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Progress in Wastewater Disinfection Technology

Proceedings of the National Symposium Cincinnati, Ohio September 18-20, 1978

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16. ABSTRACT

This symposium brought together scientists, consulting engineers, municipal design engineers, and various technical, state, and local government officials to listen to presentations on the latest developments in the field of wastewater disinfection and to actively participate in floor discussions on the data and interpretations presented therefrom.

Rapid progress is being made in the field of wastewater disinfection, but much more work is needed before a design manual can be formulated. It appears that considerable savings in chlorine usage is possible with a well designed, optimized mixing and contacting system. Dechlorination with sulfur dioxide is cost-effective, but not with activated carbon or holding lagoons. Disinfection of well oxidized, filtered secondary effluent is best achieved with a bubble diffuser contactor and is independent of contact time. Total costs of ozone disinfection appear to be twice the cost of chlorine. Ultraviolet light is finally being recognized as an extremely effective alternative disinfection process, with costs ultimately promising to be competitive with chlorine. Chlorine dioxide is somewhat disappointing compared with chlorine on bench-scale analysis, but pilot testing should reveal the true effectiveness. Very interesting preliminary results were presented on nonvolatile organic by-product formation by chlorine, ozone, and ultraviolet light. This report is of interest to consulting engineers, municipalities, and research scientists pursuing research in disinfection technology.

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PROGRESS IN WASTEWATER DISINFECTION TECHNOLOGY

PROCEEDINGS OF THE NATIONAL SYMPOSIUM
CINCINNATI, OHIO
SEPTEMBER 18-20, 1978

Edited by

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Wastewater Research Division
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Cincinnati, Ohio 45268

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
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CINCINNATI, OHIO 45268

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic efforts of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

The most important aspect of any successful research endeavor is the proper transfer of the new or developed technology to the user community. A research symposium is one mechanism for achieving that objective quickly and accurately. The proceedings reported herein is a concise presentation of the latest results from research in wastewater disinfection technology.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

This symposium brought together scientists, consulting engineers, municipal design engineers, and various technical, state, and local government officials to listen to presentations on the latest developments in the field of wastewater disinfection and to actively participate in floor discussions on the data and interpretations presented therefrom. The symposium was divided into several main sessions, each session being limited to a specific topical area. Following each formal presentation, a 5 minute informal question and answer period was allotted. Following each session, a 30 to 75 minute panel or round table discussion was held in order to permit maximum audience participation and complete thought development. This turned out to be a great success, as many thoughts flowed forth from the minds of many people who may not have participated without the proper time allotment. Some questions were answered, but many more were raised, which is what should happen at a gathering of research scientists and engineers actively involved in public health and environmental research.

Rapid progress is being made in the field of wastewater disinfection, but much more work is needed before a design manual can be formulated. It appears that considerable savings in chlorine usage is possible with a well designed, optimized mixing and contacting system. Dechlorination with sulfur dioxide is cost-effective, but not with activated carbon or holding lagoons. Disinfection of well oxidized, filtered secondary effluent is best achieved with a bubble diffuser contactor and is independent of contact time. Total costs of ozone disinfection appear to be twice the cost of chlorine. Ultraviolet light is finally being recognized as an extremely effective alternative disinfection process, with costs ultimately promising to be competitive with chlorine. Chlorine dioxide is somewhat disappointing compared with chlorine on bench-scale analysis, but pilot testing should reveal the true effectiveness. Very interesting preliminary results were presented on nonvolatile organic by-product formation by chlorine, ozone, and ultraviolet light.

A brief comment concerning organization of the proceeding's contents is in order. The papers are printed in exactly the same order they were presented, except for the banquet speech. However, most of the printed material appears in much greater detail than was presented orally. All extemporaneous discussions were reproduced with as much fidelity as possible from the original tapes. In transferring the spoken word to the written word, I exercised a certain amount of editorial prerogative in the interest of maintaining conciseness and clarity without changing the thought or intent of the statement.

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Special appreciation is given to the speakers and authors of the papers for their many hours of labor and preparation, and to the general registrants whose lively participation in the panel and round table discussions contributed greatly to the success of the symposium.

SECTION 1. OPENING REMARKS

1.

OPENING SESSION WELCOME

Albert D. Venosa
Research Microbiologist
Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio

It is a pleasure to welcome all of you to the U.S. Environmental Protection Agency's National Symposium on Wastewater Disinfection. This symposium is being sponsored by USEPA's Municipal Environmental Research Laboratory (MERL) of the Office of Research and Development, and the Environmental Research Information Center, Cincinnati, Ohio. The objectives and goals of the symposium will be presented in a few moments by Mr. John J. Convery, Director of MERL's Wastewater Research Division.

I would like to highlight the symposium's program, of which all of you should have a copy. We have organized the program into seven major sessions, each of which deals with a specific topic or aspect of wastewater disinfection research. In the first session you will be introduced to the Acting Deputy Assistant Administrator of the Office of Air, Land, and Water Use, Dr. Courtney Riordan, and MERL's Wastewater Research Division Director, Mr. John J. Convery. The subsequent sessions will be technical in scope and will involve investigators who have been granted EPA funds to conduct wastewater disinfection research. Each of the first four sessions will deal exclusively with a specific disinfectant. Thus, Session 2 concerns chlorination-dechlorination, Session 3 chlorine dioxide, Session 4 ultraviolet light, and Session 5 ozone. Session 6 involves two studies which support all the disinfection projects, i.e., indigenous virus inactivation and nonvolatile organic compound formation by chlorine, ozone, and UV light. The final session was included to provide insight into how planning decisions on implementation of new disinfection technology are made at both the EPA Regional level and the municipal consultant's level. In order to permit the maximum amount of audience participation, each session will be concluded by a separate panel or round table discussion. You are encouraged to air your views to the fullest during these discussions.

Following the last panel discussion, Dr. E. J. Middlebrooks, Dean of the College of Engineering, Utah State University, an able environmental researcher himself, will summarize the findings of the symposium and provide perspective into what lies ahead for EPA's Wastewater Disinfection Program.

On Tuesday evening, there will be a social hour and banquet, beginning at 6:00 P.M. I am certain you will enjoy listening to the guest speaker, Mr. Joseph V. Karaganis, the Special Assistant Attorney General of the State of Illinois, who will discuss the microbial aspects of the case in which he won a landmark decision for the State of Illinois against The City of Milwaukee. The title of his paper is "The Law's Response to Public Health Hazards."

All presentations and panel discussions will be recorded and published in a proceedings. I hope you enjoy the symposium and encourage you all to participate actively.

2.

INTRODUCTION AND OBJECTIVES OF SYMPOSIUM

John J. Convery
Director, Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio

On behalf of the Municipal Environmental Research Laboratory of the U.S. Environmental Protection Agency, I welcome you to this National Symposium on Wastewater Disinfection. The basic purpose of this symposium is to share with you the most up-to-date research findings of our disinfection research program and thereby accelerate the practical application of these findings to the construction grants program. A corollary objective is to obtain feedback from you on your ideas or problems which can be incorporated into our future research program. The Office of Research and Development of EPA wants to be responsive to the needs of you, the practitioners of pollution control technology.

During the next three days there will be opportunities for audience participation. I would like to second Al Venosa's invitation to participate in these discussions. If additional thoughts on disinfection research needs occur to you after the symposium is finished, please share them by writing to me.

This is the second effort of the disinfection research staff to share timely research findings through the medium of a public meeting. In October 1974, we held a workshop in Wyoming, Michigan, where the results of continuous fish bioassays on chlorinated, chlorinated/dechlorinated, ozonated, and bromine chloride treated wastewaters were presented. The primary interests at that time were the relative degrees of disinfectant-induced fish toxicity and the methods of reducing or eliminating the toxicity. The prevention of fish toxicity is still an important design objective and a fundamental reason for our interest in improved chlorination and dechlorination. The current EPA criteria for total residual chlorine to prevent fish toxicity is 2.0 µg/l for salmonid fish and 10.0 µg/l for other freshwater and marine organisms (3).

Several events have occurred since 1974 which make the task of developing acceptable disinfection technology more difficult for all of us. Widespread occurrence of the formation of trihalomethanes in chlorinated waters and recognition of their potential health effects together with passage of the Toxic Substances Act (PL94-469) and the signing by EPA of the consent degree (2) to control 129 priority pollutants within 3 years, have added analytical chemistry and toxicological screening requirements to our evaluation of disinfection alternatives. This is particularly true for potential reuse situations. A significantly expanded analytical methods development program is underway to permit surveying the occurrence of priority pollutants in municipal raw wastewaters, process influents and effluents, and sludges. Recommended analytical methods should be available by the first of November, 1978. The Health Effects Research Laboratory, EPA, Cincinnati, is currently evaluating a variety of toxicological screening tests which may be useful in evaluating the efficacy of alternative disinfectants. A recent paper by Bull, Kopfler and McCabe (1) describes these tests, which I have listed for your information.

ACUTE TOXICITY	-MEDIAN LETHAL DOSE (LD_{50})
MUTAGENICITY	-AMES TEST: *Salmonella typhimurium* strains (bacteria) SRI TEST: *Saccharomyces cerevisiae* (yeast)
CARCINOGENICITY	-*IN VITRO*

	MAMMALIAN CELL CULTURE WITH TRANSFORMED CELLS INJECTED INTO MICE TO NOTE TUMOR PRODUCTION
	-SKIN TUMOR RESPONSE IN SENSITIZED MICE (SEN-CAR MOUSE)
	-*IN VIVO* ASSAY USING NEONATAL RATS
TERATOGENICITY	-FEEDING OF PREGNANT RATS AND NOTING MORPHOLOGIC CHANGES IN THE NEW-BORN

Since 1974, the disinfection program has spent approximately $3 million on projects to develop and test alternative disinfection approaches. Many of the facilities involved are "among the first of their kind" treatment plants. In this category I would include: the Estes Park, Colorado, ozonation with air facility; the Meander, Ohio, ozone with oxygen facility; and, the Northwest Bergen County ultraviolet light treatment facility. Obviously, most of our limited resources are used to monitor and evaluate performance rather than pay for capital facilities.

The amendments to the Clean Water Act (PL95-217) which passed last Fall, include a provision for innovative and alternative process or system designs which permits, after October 1, 1978, 85% construction grant financing of the capital requirements and provides for a 100% replacement or modification guarantee. To qualify as innovative technology, a process must save 15% of the total life costs or 20% of the energy requirements compared to conventional technology. This provision illustrates two other significant elements of concern in comparing disinfection alternatives-energy utilization and total treatment costs. Comparisons cannot be made accurately unless the same endpoint or treatment objective is stated. This brings me to my final point. In reading through disinfection literature I am always impressed with the number of variables that need to be recognized, and the necessity of their measurement or identification to permit meaningful comparison of results. The following list may not be all inclusive but it serves to illustrate my point.

DISINFECTANT — type (chlorine), form (monochloramine)

DOSAGE — APPLIED - DISINFECTANT DEMAND = EFFECTIVE DOSAGE

QUALITY OF THE FEED WATER (AND COST IF A NECESSARY PRETREATMENT REQUIREMENT FOR A PARTICULAR DISINFECTANT)—Suspended solids, color, COD, pH, NO_2, NH_3, H_2S.

BACTERIOLOGICAL QUALITY MEASUREMENT — indicator (coliform, MPN, MF), pathogen (bacteria or virus [wild or cell cultured]), density objective, point of measuring effluent quality, opportunity for regrowth or photoreactivation, influent density effect on % removal vs. number remaining.

CONTACTOR — mixing conditions (GT), type (pressure or open), geometry (length to width ratio), real vs. theoretical detention time.

My purpose in raising the issue of variables and comparable treatment objectives is to encourage more complete descriptions of your evaluations and thereby permit more meaningful comparisons and the rational assignment of pre- and post-disinfection process costs when comparing disinfectants. Examples might be the cost of chemical clarification and filtration to meet a 2.2 MPN/100 ml coliform disinfectant objective or the reaeration costs to meet a dissolved oxygen requirement for a dechlorination facility.

I hope you experience an enjoyable and worthwhile symposium.

REFERENCES

1. Bull, R. J., et al. 1978. Toxicity and Mutagenic Effects of Organics. Presented 1978 Annual AWWA Conference, Atlantic City, N.J.
2. *Natural Resources Defense Council* vs. *Train.* 8 ERC 2120 (D.D.C. 1976).
3. Quality Criteria for Water. 1976. U.S. Environmental Protection Agency, Washington, D.C. 20460.

DISCUSSION

DR. HARVEY ROSEN, *Union Carbide Corporation:* I would just like to reiterate your last point. I think it is very important.

Recently, in a National Academy of Sciences study, investigators had been trying to collect information to do meaningful comparisons among different disinfectants as well as comparisons using a single disinfectant in different quality waters. They are discovering that, although there are thousands and thousands of reports dealing with these studies, lack of quality in the reporting is evident. Certain specific information, needed to determine the validity of the results, is missing. This is becoming very important in terms of all the money that is being spent in this area since results that can be used and understood by everybody on a common basis are not available.

3.

PERSPECTIVES ON WASTEWATER DISINFECTION - A VIEW FROM HEADQUARTERS

Dr. Courtney Riordan
Acting Deputy Assistant Administrator
Office of Air, Land, and Water Use
U.S. Environmental Protection Agency
Washington, D.C. 20460

Before beginning my prepared remarks, I would like to second the welcome to the Symposium.

It really warms my heart to see all these people here because one of the most sensitive areas that we deal with in headquarters in terms of our research program in EPA is our delivery system. That is the one that is constantly questioned. By that I mean, how do we interact with our user groups? How do we get feedback from our user groups and ultimately influence the character of our research program?

Looking at the group we have convened today and studying the program, I think it is going to be a very productive conference. It gives me the kind of first-hand experience that greatly facilitates returning to Washington and pointing out that we do have some fairly effective mechanisms for communicating our programs to the public.

On July 26, 1976, the U.S. Environmental Protection Agency made the decision to delete the fecal coliform bacteria limitations from the definition of secondary treatment (40 CFR 133). The intent was to permit reliance on site-specific water quality standards to establish disinfection requirements for municipal wastewater treatment plants rather than mandate continuous, year-round disinfection of *all* effluents. The detailed rationale behind this decision has been published and need not be discussed today. It is important to consider, however, the impact this decision has had both on the Municipal Environmental Research Laboratory's (MERL) Wastewater Disinfection R&D Program and on the individual states' reactions.

Public Law 95-217 states that, wherever attainable, water quality which protects fish, shellfish, and wildlife and provides for recreation will be achieved by July 1, 1983. Two important questions relative to the 1983 goals are raised in connection with the disinfection deletion from the Secondary Treatment Regulation. First, is it possible to comply with the 1983 goal of swimmable waters without universal, year-round disinfection? Secondly, how can the goal of providing for the protection and propagation of fish, shellfish, and wildlife be satisfied when disinfection is required to maintain conditions suitable for recreation?

Water quality standards are presently being reviewed and revised to define conditions necessary to meet the 1983 goal. Reliance on state water quality standards in lieu of fecal coliform bacteria limitations in the Secondary Treatment Regulation is consistent with the achievement of the 1983 goal, because water quality standards are established, in part, to allow recreation in and on the water. Implementation of water quality standards for recreation in and on the water can result in the following types of situations:

(a) Where water quality standards are established to allow whole body contact recreation, disinfection of wastewater discharges will generally be necessary to meet bathing water criteria, such as 200 fecal coliform bacteria per 100 ml over a 30-day period. In instances where bathing waters are used seasonally, water quality standards for whole body contact may not apply during non-use periods and, consequently, year-round wastewater disinfection may not be required.

(b) Where water quality standards are established for secondary contact recreation purposes such as boating, in-stream criteria may be as high as 1000 to 5000 fecal coliform bacteria per 100 ml. In these instances, wastewater disinfection may not always be needed to meet the required in-stream standards.

(c) In accordance with regulations pertaining to the establishment of water quality standards (40 CFR 130), the states may, under certain prescribed conditions, set standards at levels less stringent than the national water quality goal. Wastewater disinfection will generally not be necessary in these instances.

Similarly, water quality standards for total residual chlorine can be established to address the potentially adverse effect of chlorination on aquatic plants and organisms. In situations where the needs for disinfection and protection of aquatic organisms co-exist, several alternatives are available. First, sufficient mixing and dilution may be available to meet instream chlorine criteria, particularly where chlorination processes are properly designed and operated. Dechlorination is also available to eliminate the residual chlorine from disinfected effluents prior to discharge. Finally, alternative disinfectants, such as ozone or ultraviolet light, may prove to be cost-effective where stringent effluent standards are specified.

As achievement of Public Law 95-217 by the 1983 goal becomes imminent, the Agency will be in a better position to re-evaluate the disinfection requirements for municipal wastewater discharges in consideration of the improved water quality at that time. In the interim, time will be available for further investigation of cost-effective disinfection processes and analysis of more conclusive data on the relative benefits and risks resulting from wastewater disinfection.

Individual States Reactions

When the decision had been made to delete the fecal coliform limitation, it was intended that the states' water quality criteria would control disinfection policy on a case-by-case basis. In some instances the criteria established for some steams have been too lax, while in others too strict, relative to the probable transmission of communicable disease through direct human contact with contaminated waters.

One such state, the State of New York, has established 14 different stream classifications based on water quality relative to use and proximity of human contact or shellfish contact with contaminated discharges. There are five separate classifications for fresh surface waters, four classifications for marine waters, four special classifications related to specific locales, and one classification established for enjoyment of water in its natural conditions. For each of these classifications a separate coliform standard is established with standards ranging from 0 discharge to a high of 10,000 total coliforms per 100 ml. It is hoped that other states will follow suit in this regard.

MERL Wastewater Disinfection R&D Program

The MERL Wastewater Disinfection R&D Program is essentially a process development effort in which the major responsibility is to develop alternative technology useful to design engineers and municipalities to decide how best to achieve a stated microbiological discharge limitation. Elimination of the Federal requirement for disinfection of secondary effluent has led to the additional responsibility for the MERL Wastewater Disinfection Program to develop the capability of the alternative technology to achieve different levels of effluent bacterial numbers rather than a fixed national goal of 200 fecal coliforms/100 ml. Depending on the water quality standard of the receiving stream, effluent limitations may range from as high as 5000 fecal coliforms/100 ml to as low as 2.2 total coliforms/100 ml. Thus, decisions as to which alternative technology to implement in a particular receiving stream could be significantly affected by the microbial limitations established for it.

Research efforts of the MERL Wastewater Disinfection Program have demonstrated the process requirements necessary to achieve flexible coliform limitations. Some of the data will be presented at this symposium. Other data will be available in the upcoming months. The Municipal Environmental Research Laboratory has implemented a combination of inhouse and extramural expertise for research and development to meet these objectives. This symposium is a hallmark to that effort.

SECTION 2. CHLORINATION/DECHLORINATION

4.

FIELD STUDIES IN OPTIMIZATION OF WASTEWATER CHLORINATION

E. Sepp, P. Bao and A. Custodio
California Department of Health Services
Berkeley, California

INTRODUCTION

California Department of Health Services is conducting field studies on optimization of wastewater chlorination. The project is partially financed by the U.S. Environmental Protection Agency (EPA) and is a joint venture with the Department of Fish and Game and State Water Resources Control Board. Three mobile units participate in the study; a chlorination pilot plant, a fish bioassay laboratory and a public health laboratory. Results from the pilot plant are compared to results obtained from parallel studies on the full scale plants. Performance is measured by means of coliform bacteria and chlorine residual tests, as well as various chemical analyses.

Objectives of the project are: reduction of chlorine induced toxicity; savings in chlorine use by improved design; and, writing of a design manual.

MATERIALS AND METHODS

The chlorination pilot plant incorporates optimized features including rapid initial mixing, automatic control of chlorine dosage by residual control, and plug flow contact chambers providing a variety of contact times. Samples are analyzed for total coliform bacteria using 5 tubes per dilution. All analyses are done according to Standard Methods for the Examination of Water and Wastewater (1). Bacteriological and chemical sampling is confined to the 8-hr. daytime period. Flow-through bioassays, done by Department of Fish and Game, are run continuously for 96 hours.

RESULTS AND DISCUSSION

The first study was done at an activated sludge plant at San Leandro. The plant flow consists of almost 50% industrial wastes and the plant is overloaded. The chlorination system consists of a manually controlled chlorinator and a spirovortex chlorine contact tank. Chlorine is injected through a diffuser into a pipe about 30m (100 feet) ahead of the contact tank. The pipe is under pressure; therefore, initial mixing probably is good. The BOD of the effluent varied from 12 to 29 mg/l, and COD from 69 to 109 mg/l. The coliform levels were extremely high.

The first study was beset with problems. During maximum flow periods there was a heavy carryover of sludge and grease from the final clarifier which clogged up the pilot plant. Consequently, the pilot plant could be operated only part-time. Bacterial results from the pilot plant were not remarkably better than those from the full scale plant. This was inexplicable and disappointing because the contact tank had a t_i^* less than 10 minutes and suspended solids content was high (32-102 mg/l). Approximately the same coliform bacteria levels were reached with chlorine dosages of 10-11 mg/l in the pilot plant and 10-12 mg/l in the full scale plant. Chlorine residuals were 5-6.5 mg/l and 6-11 mg/l, respectively, in the two plants after the same theoretical contact times.

San Pablo has an activated sludge plant which also uses a roughing trickling filter and complete nitrification. The plant receives appreciable amounts of industrial waste. Effluent quality is good, with a BOD of 4-5 mg/l and suspended solids < 10 mg/l most of the time. Furthermore, the quality is fairly constant and the coliform bacteria levels are low. The chlorination system consists of a compound loop chlorine control system, a hydraulic jump for initial mixing and two parallel rectangular baffled chlorine contact tanks with a L/W ratio of 15:1. The contact tank has a t_i of about 30 minutes. Effluent is dechlorinated with SO_2.

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*t_i is the time (minutes) for the initial appearance of dye in a dye study.

Bacterial results indicated that the same level of disinfection was obtained with a slightly lower chlorine dosage in the pilot plant than in the full scale plant. The chlorine dosages were 8-9 mg/l for the pilot plant and 9.5-10.5 mg/l for the full scale plant. Chlorine residuals at the end of contact were about 1.5-2.0 mg/l for both plants. The detention time in the pilot plant was slightly smaller than in the full scale plant.

The Pinole plant uses activated sludge for treatment. Chlorination system consists of compound loop chlorine control system, two turbine mixers and two parallel chlorine contact tanks with a L/W ratio of 40:1. The waste is entirely domestic. However, the chlorine flow to each tank is poorly balanced and during low flows at night the chlorine residual drops to zero. Furthermore, there is leakage through the baffles in the contact tank resulting in a t_i of only about 15 minutes. Effluent is dechlorinated with SO_2. The effluent BOD varied from 8 to 29 mg/l, and suspended solids from 2 to 60 mg/l. There was considerable amount of nitrification, and nitrite content was fairly high (1.0-2.9 mg/l). The chlorine demand usually was very high in the morning and dropped to a low level in the afternoon. The coliform bacteria content varied from day to day and week to week.

Due to the varying effluent quality, the results were difficult to analyze; however the chlorine dosages to achieve the same bacteria levels were 1.5-3 times higher for the full scale plant than for the pilot plant. The chlorine residuals were 3.4-3.7 mg/l for the pilot plant and 5.5-13.6 mg/l for the full scale plant after detention. The corresponding dosages were 7.5-27.7 mg/l for the pilot plant and 27-47 mg/l for the full scale plant.

Pilot plant studies were also done at each plant using different chlorine dosages to obtain different effluent bacteria levels. An MPN level of 2 per 100 ml after 60 minutes contact was achieved with the following chlorine dosages and control residuals, respectively: 22 mg/l and 11.5 mg/l at San Leandro; 10-13 mg/l and 7-8 mg/l at San Pablo; and 16.2-22.8 mg/l and 4.6-5.5 mg/l at Pinole (one week only). An MPN level of 23 per 100 ml after 30 minutes contact was obtained with the following dosages and residuals: 22 mg/l and 11 mg/l at San Leandro; 10 mg/l and 7 mg/l at San Pablo; and 5.7-8 mg/l and 2-3.7 mg/l at Pinole.

Various authors (2,3) have indicated that bacterial survival ratio is dependent on the product of chlorine residual and detention time. Collins (2) developed the following empirical formula:

$$N_1 / N_0 = \left[\frac{(R_0 T)}{a}\right]^b$$

where N_1 = bacteria number in chlorinated effluent
N_0 = bacteria number in unchlorinated effluent
R_0 = chlorine residual, mg/l, after mixing (the "control" residual)
T = detention time, minutes
b = slope of the curve
a = a coefficient

This formula plots as a straight line on log-log paper. Collins found that the slope b is -3 for plug flow and -1.5 for completely mixed flow. The ratio N_1/N_0 is the bacterial survival ratio. In this study the residual after contact was designated as R, without subscript.

Plots of bacterial survival ratio against RT for pilot plant and for full scale plant for the three studies are shown in Figures 1 through 6. The solid line represents the theoretical plug flow curve. The data points are daily median values. The pilot plant data include variation in detention times from 15 to 60 minutes. Computer regression analysis yielded the following values:

	a	b	r*
San Leandro plant	30.1	-1.53	0.68
Pilot plant at San Leandro	5.7	-3.3	0.77
San Pablo plant	0.1	-1.1	0.43
Pilot plant at San Pablo	14.6	-4.1	0.93
Pinole plant	0.2	-1.35	0.40
Pilot plant at Pinole	0.16	-1.71	0.66

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*r = correlation coefficient.

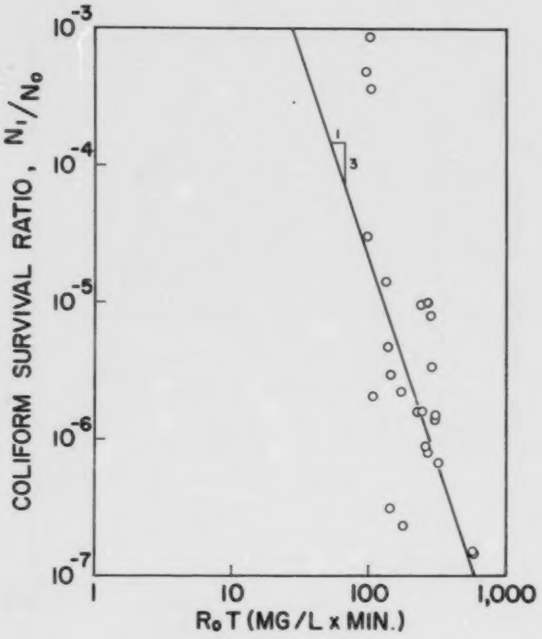

Figure 1. Coliform Survival in Pilot Plant (San Leandro)

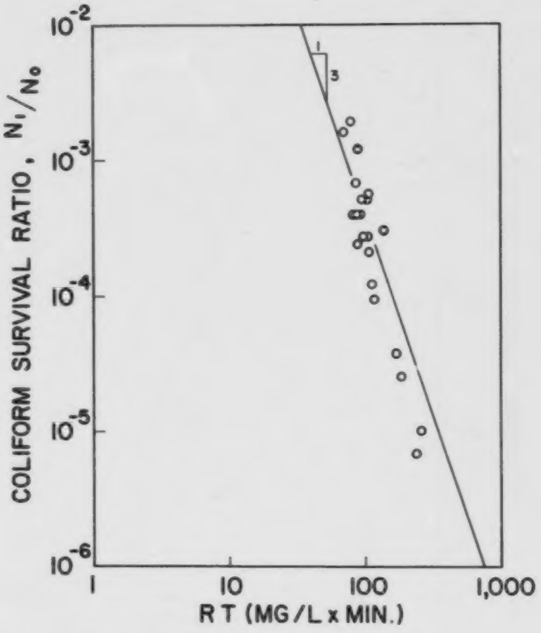

Figure 3. Coliform Survival in Pilot Plant (San Pablo)

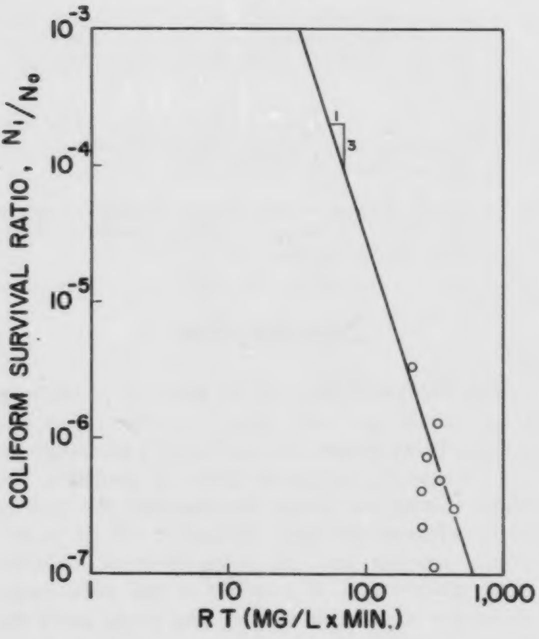

Figure 2. Coliform Survival in San Leandro Plant

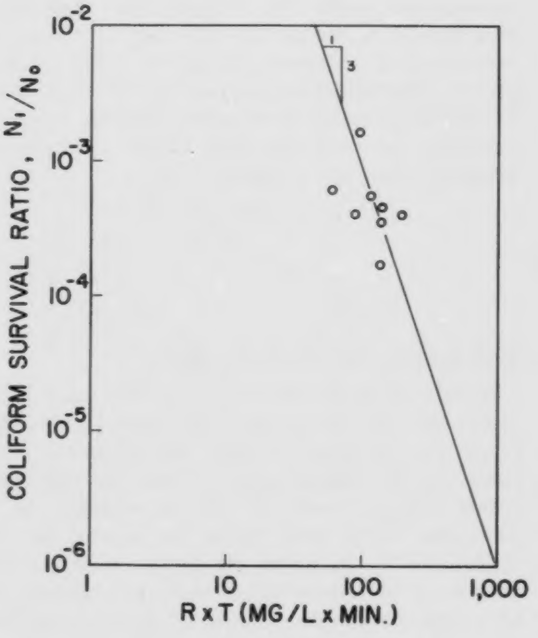

Figure 4. Coliform Survival in San Pablo Plant

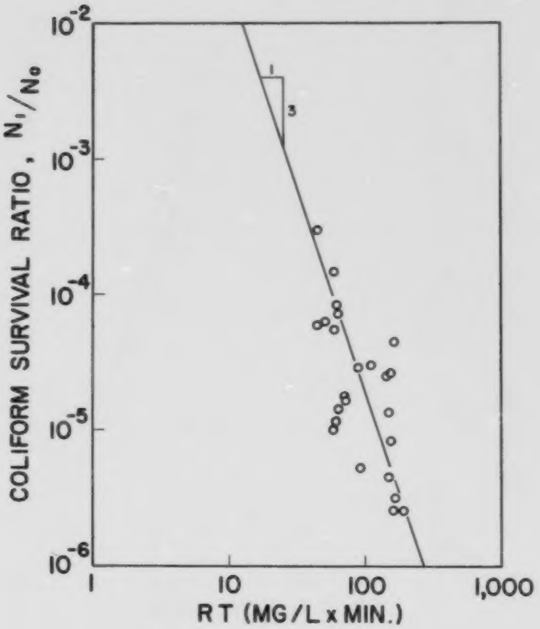

Figure 5. Coliform Survival in Pilot Plant (Pinole)

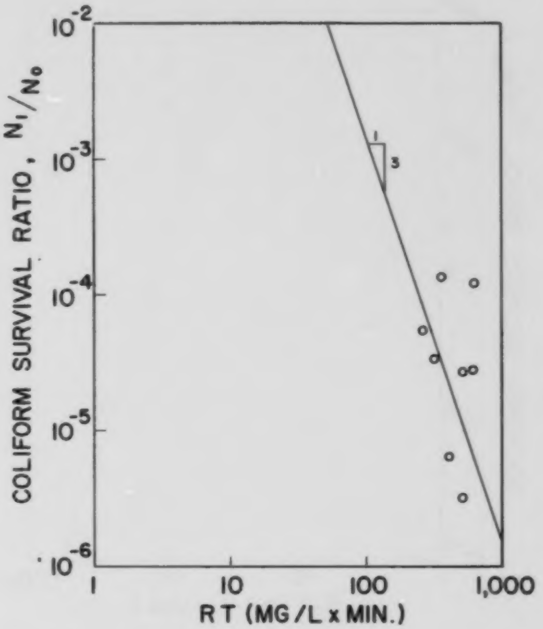

Figure 6. Coliform Survival in Pinole Plant

The regression curves for the pilot plant differ considerably from the −3 slope for theoretical plug flow. One reason for this may be the rapid variations in effluent quality at some of the plants. The regression curves for the full scale plants have small correlation coefficients (r) reflecting the fact that only small variations in detention times are available.

SUMMARY AND CONCLUSION

Results of three chlorination studies have been discussed. At two plants, the optimized pilot plant was able to disinfect the effluents with lesser chlorine dosage required than the full scale plants. Dosages and residuals to disinfect to a particular MPN level appear to depend on effluent quality, including the initial coliform bacteria densities. Bacterial survival ratio appears to be a function of the product of chlorine residual and contact time, but is further influenced by changes in effluent characteristics.

REFERENCES

1. American Public Health Association. 1975. Standard Methods for the Examination of Water and Wastewater, 14th ed. American Public Health Association, Inc., New York.

2. Collins, H.F., and Selleck, R.E., "Process Kinetics of Wastewater Chlorination." SERL Report No. 72-5, Univ. of California, Berkeley, (Nov., 1972).

3. Sanitation Districts of Los Angeles County, "Pomona Virus Study," California State Water Resources Control Board, Sacramento, (June, 1977).

DISCUSSION

MR. GEORGE WHITE: It would be a big help if we could get you scientists, when you are making those graphs, to put a ball park range on N_1 because N_1 coliform count is probably the primary criterion which demonstrates the quality of the effluent and how difficult it will be to disinfect, whether you are using chlorine or some other disinfectant. If you had a ball park range on each one of those graphs, you could apply the formula. This is the first thing you have to know when you are trying to get down into the

240/100 range or the 23 or the 2.2 range . . . You have to know what the initial number of the coliforms is before you hit it with the disinfectant. If you could just put those ball park figures up in the lefthand corner of those graphs it would be very helpful.

MR. SEPP: I think this is a very good point. Coliform bacteria content seems to be the main criterion.

MR. OPATKEN: What is your definition of low levels of coliforms? Somewhere during presentation of your paper you referred to a low level of coliforms.

MR. SEPP: Low level was 100,000 to 500,000 per 100 ml.

MR. VENOSA: Were the three treatment plants that you studied capable of meeting the coliform standard that was established for those particular plants, and was the pilot plant able to do better?

MR. SEPP: Yes, but they over chlorinated because usually their effluent levels were very low. All their requirements were 240 MPN/100 ml.

MR. VENOSA: All three plants?

MR. SEPP: Their actual levels were more like 10/100 ml.

MR. VENOSA: If I remember correctly, it was at the Pinole plant that only about a 4 log reduction was achieved, and I was wondering how many organisms that represented?

MR. SEPP: It varied . . . sometimes it was around 50,000, other times there were around 10,000,000. It varied through that range and each week there was some change. It was partly caused by use of the community. When school was out, bacteria levels dropped. When school was in, bacterial counts were higher.

DR. HARVEY ROSEN: I would like to say that we reached similar conclusions with ozone. I am pretty sure you will find that with most disinfectants the initial coliform count is extremely important in determining the amount of disinfectant, assuming that you have a well mixed reactor and other things. The exception is when you are designing a system; then, it is very difficult to know what the expected level of coliforms is going to be. From a practical point of view you then have to relate coliform levels to effluent quality. The better the effluent quality, generally, the lower the level of coliforms. From a point of view of design, you then have to generally know something about or make certain assumptions about the treatment processes that, preceding disinfection, will produce a specific BOD, suspension solids, and COD in order to estimate and evaluate a specific dose of disinfectant. It would be nice if we were able to pilot all of these things and know what kind of coliform count we expected from these various treatment processes. That is the best index, as we have found, although you do need some other things as tools.

MR. WHITE: This goes back to the same thing that I said about putting the range of the coliforms on the graphs. This is one of the things that I tried to do when I was writing the second book which, incidentally, will be out in October. I have it all in there so it will answer Harvey Rosen's question. I collected as much data as I could about that N_1 from all the different types of effluents. I just put it down in a range for well oxidized secondary, for primary, which I do not think you can disinfect, and for tertiary with and without sedimentation and filtration. It is all in there and, of course, the more data we collect, the better it will be. Good data is very hard to come by.

I might also add that one thing people overlook when they start talking about disinfection of wastewater, is that the operator has to have all of his unit processes working before the disinfection system will really take over. This is why disinfection, if nothing else, is a good monitor of how well the operator is operating his plant.

5.

COMPARISON OF ACUTE TOXICITY OF CHLORINATED EFFLUENTS FROM OPTIMIZED AND EXISTING FACILITIES

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ABSTRACT

The California Department of Health in cooperation with the California Department of Fish and Game developed and implemented a chlorine optimization study which investigated several design criteria that may improve the efficiency of wastewater chlorination systems, and hence, provide adequate disinfection without excessive chlorination. The study has been conducted on site at several wastewater treatment plants in northern California. Two mobile units were constructed for the project; a pilot chlorination plant and a mobile water quality and bioassay laboratory were operated by the Department of Health and the Department of Fish and Game, respectively. The pilot chlorination plant tested several optimized chlorination design criteria against existing wastewater treatment plant chlorination systems. The bioassay laboratory tested the toxicities of the optimized and existing chlorinated effluents.

The results obtained thus far indicate that no toxicity was associated with either the existing unchlorinated or dechlorinated effluents. The optimized chlorinated effluents had lower and more stable chlorine residuals than did the existing chlorinated effluents and hence, were less toxic. The toxicity of all effluents investigated increased proportionately with increased chlorine residual. The toxicity of chlorine in wastewater was lowered when denitrification was practiced prior to chlorination.

INTRODUCTION

The toxicities of chlorine and chlorinated wastewater effluents to fish and other aquatic life are well documented. Many investigators [Zillich (20); Arthur *et al* (2); Brungs (4,5); Ward *et al* (19); Mattice and Zittel (12); and Finlayson and Hinkelman (10)] have clearly demonstrated that chlorine is highly toxic to most aquatic life. Lethal concentrations of chlorine to fish falls between 23 µg/l (0.023 mg/l) for rainbow trout *(Salmo gairdneri)* and 500 µg/l (0.5 mg/l) for threespined stickleback *(Gasterosteus aculeatus)*. The U.S. EPA (18) has recommended that "safe" chlorine residuals in the environment should not exceed 10 µg/l (0.01 mg/l) for the protection of aquatic life. Even lower levels of chlorine than those recommended have been reported to impair growth and development of some aquatic biota [Roberts *et al.* (15)].

The disposal of chlorinated wastewater effluents in the environment has created a long-standing conflict between public health and conservation agencies. Health agencies are concerned with the adequate disinfection of wastewater effluents for the protection of public health; whereas conservation agencies are responsible for protecting the aquatic resources in the environment. Section 5650 of the State of California Fish and Game Code (1977) prohibits the discharge of any substance deleterious to fish, plant or bird life into the waters of the State; consequently, confrontations between public health agencies and conservation agencies over the discharge of chlorinated wastewaters are common.

Our primary concern with the chlorination of wastewater has been that chlorine is not always carefully and efficiently applied at wastewater treatment plants. We have observed the discharge of effluents into the environment with total residual chlorine (TRC) concentrations in excess of 10 mg/l. This level is 1000 times the recommended "safe" level. Usually, excessive chlorine residuals in a discharged waste are the result of improper chlorine application, an ineffective residual chlorine control system, or both.

Because of this need to optimize wastewater chlorination systems, we welcomed the opportunity to participate in a Chlorination Optimization Study which may help to minimize chlorine residuals discharged into the environment. We point out, however, successful chlorine optimization only solves a portion of the problem. Chlorinated effluents usually must be subjected to some form of dechlorination prior to discharge so that adequate protection is afforded to the biota using the receiving water.

The purpose of the Chlorination Optimization Study is to evaluate the influence and importance of three chlorine application optimization criteria. These design criteria originally developed by Collins and Deaner (6) are:

1. rapid and complete initial mixing between chlorine and waste;
2. a 30-min minimum contact time between the chlorine and waste with no further mixing; and
3. a sound and workable chlorine residual control system.

The Department of Health (DOH) has built a mobile chlorination pilot plant for the study and has used the plant to test the chlorine application optimization criteria against existing wastewater chlorination facilities. The Department of Fish and Game's objective in the study is to document the comparative toxicities between the optimized and existing chlorinated effluents from several different wastewater treatment facilities under study by DOH. We accomplished this by means of a specially designed mobile laboratory capable of simultaneously monitoring the water quality and toxicity of three separate effluents:

1. existing WTP unchlorinated (UnCl) effluent;
2. existing WTP chlorinated (ECl) effluent; and,
3. DOH optimized pilot plant chlorinated (DOHCL) effluent.

During periods when the DOH pilot plant was not in operation, toxicity and water quality data were also collected from existing dechlorinated (DeCl) effluents.

To date, both units have participated in comparative studies at several wastewater treatment plants (WTP) in California. Since the project is on-going, this paper is only a progress report. The final report will not be available until after June 1979. Our present analyses of the data demonstrate the differences between:

1. magnitude and variability of chlorine residuals in optimized and existing chlorinated effluents;
2. toxicity of optimized and existing chlorinated effluents;
3. toxicity of chlorine in ammoniated and ammonia-stripped (denitrified) effluents; and,
4. toxicity of chlorine to the two test fish, fathead minnow *(Pimephales promelas)* and golden shiner *(Notemigonus crysoleucas).*

MATERIALS AND METHODS

Project Schedule

To date, both units have been in operation at four wastewater treatment plants in northern California (Table 1). We have collected comparative toxicity data between optimized and existing chlorinated effluents for only two plants, San Pablo and Pinole. The pilot plant was not in operation at San Leandro and although recent comparative toxicity data are now available from the South San Francisco facility, they have not been analyzed yet.

Mobile Laboratory

The DFG mobile water quality and toxicity

TABLE 1. SCHEDULE FOR BIOASSAY TESTING AT VARIOUS WASTEWATER TREATEMENT PLANTS IN CALIFORNIA DURING THE PERIOD OF FEBRUARY THROUGH AUGUST 1978.

Plant Location	Date Begin	Elluent Type [a]	Test Species [b]	Series No.
San Leandro	Feb. 13, '78	UnCl, ECl	FH	SL-1&2
	Feb. 27, '78	UnCl, ECl	FH	SL-3&4
San Pablo	Apr. 10, '78	UnCl, ECl, DOHCl	FH	SP-1,2,&3
	Apr. 17, '78	UnCl, ECl, DeCl	FH	SP-4,5,&6
	Apr. 24, '78	UnCl, ECl, DOHCl	FH	SP-7,8,&9
Pinole	Jun. 12, '78	UnCl, ECl, DOHCl	FH	P-1,2,&3
	Jun. 19, '78	UnCl, ECl, DeCl	GS	P-4,5&6
	Jun. 26, '78	UnCl, ECl, DOHCl	GS	P-7,8&9
	Jul. 8 '78	ECl	FH, GS	P-10&11
South San Francisco	Aug. 14, '78	ECl	FH, GS	SSF-1&2

(a) ECl = Existing chlorinated effluent
UnCl = Existing unchlorinated effluent
DOHCl = Department of Health Pilot Plant Chlorinated effluent
DeCl = Existing dechlorinated effluent

(b) FH = Fathead minnows *(Pimephales promelas)*
GS = Golden Shiners *(Notemigonus crysoleucas)*

FIGURE 1. The California Department of Fish and Game's Field Laboratory.

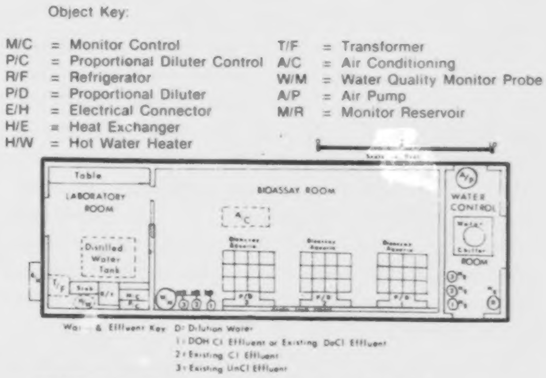

FIGURE 2. Floor plan of mobile laboratory

laboratory is an 8 by 4 by 2.7 m trailer (Figure 1). The unit is transported on site by a 1-ton stakeside truck. The total cost of the mobile laboratory, including all analytical instruments, is approximately $30,000.

The mobile laboratory is functionally segregated into three separate rooms (Figure 2): 1. Water Control Room; 2. Bioassay Room; and, 3. Laboratory Room.

The Water Control Room receives up to three wastes and the dilution water for the bioassays. Equipment in this room includes: a chiller, heat exchangers, and an air pump. There are four heat exchangers, one for each of the wastes and one for the dilution water. The heat exchangers, which have the capacity of lowering the temperatures of incoming water and wastes, are constructed of 10.2 cm and 15.2 cm diameter PVC pipe with stainless steel tubing contained inside. The exchangers work on the principal of heat transfer from the water and waste streams to chilled ethylene glycol in the steel tubing as the streams flow through the PVC pipes. The chilled ethylene glycol is supplied from a 1-ton, air-cooled, water chiller.

The Bioassay Room contains three proportional diluters [Mount and Brungs (13)] and 36 aquaria for three continual-flow bioassay tests. An air conditioner, a waste and dilution water delivery system, and an automatic water quality monitor are also present (Figure 2).

The automatic water quality monitor has four overflowing reservoirs; one for the multiparameter probe (measuring unit) and one for each of the three wastes. The monitor continually records pH, temperature, dissolved oxygen, and conductivity of each undiluted waste stream on a recycling basis. The recycling interval can be adjusted from 30-min to 12-h.

The Laboratory Room houses the electronic controls for the proportional diluters and the water quality monitor, and the various instruments needed for the chemical and physical monitoring of the bioassay aquaria during the continual-flow bioassays (Figure 2). The control unit of the water quality monitor, in addition to recording the four water quality parameters, records a code for each waste and the time-of-day on paper and cassette tapes.

The laboratory operates on standard 120/240 VAC electricity which can be supplied from two separate sources. Generally, a 3P 480 VAC source is obtained from the treatment plant and connected to the trailer. The 3P supply is converted to 1P 240/120 VAC through a 1P 480/240 VAC transformer located underneath the trailer. A standby 1P 240 VAC diesel-powered portable alternator can also be used.

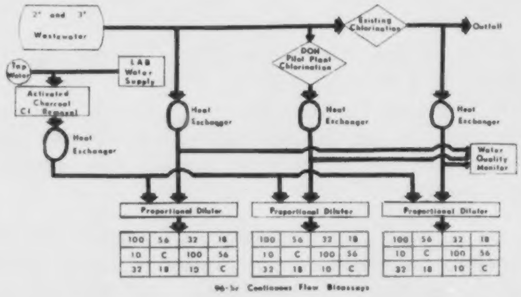

FIGURE 3. Schematic diagram of waste and water flow in mobile laboratory

The flow of water and waste through the mobile laboratory is schematically diagrammed in Figure 3. The water and waste delivery system of the mobile laboratory is constructed out of PVC SCH 40 and 80 pipe, fittings, and valves. Nylon garden hoses connected to submersible pumps supply the wastes to the trailer. Dechlorinated tap water is used as the dilution water for the bioassays. Dechlorination is accomplished by activated charcoal contained inside of the large (dilution water) heat exchanger. The flows of the wastes and dilution water in the mobile laboratory are controlled by a series of PVC ball valves (which function as shunts) located downstream of the heat exchangers. Proportions of the wastes are drained off and supplied to the water quality monitor before they enter the proportional diluters.

Bioassay Methods

Standard 96-h continual-flow bioassay methods [APHA (1); Peltier (14)] were used in the toxicity testing. Both fathead minnows and golden shiners were used as the test organisms. The proportional diluters supplied continual waste concentrations in a geometric series (100, 56, 32, 18, 100%) and a control (100% dilution water) to the bioassay aquaria during the 96-h tests. All waste concentrations and the control were done in replicate. Fifteen fish were tested in each 10 l overflowing plexiglass aquarium. The volume of each aquarium was replenished every 2.5-h. Fish mortality was determined every 24-h. Adequate dissolved oxygen was supplied to the test aquaria by aeration.

Several water quality parameters using Standard Methods [APHA (1)] were manually measured in the bioassay aquaria every six hours during the bioassays. Dissolved oxygen and temperature were measured with a daily calibrated dissolved oxygen meter and probe, and pH was determined using an expanded-scale pH meter and combination electrode. Total ammonia was determined with a specific-ion meter and NH_3 gas-sensing electrode. Total residual chlorine was measured with an amperometric titrator.

During the bioassay tests, total residual chlorine and total ammonia were manually determined at 2-h intervals in each undiluted waste stream along with the automatically measured levels of dissolved oxygen, pH, temperature, and conductivity.

Data Analysis

We estimated mortality for each waste concentration during the 96-h test from $m=(1-S_x/S_c)$ 100, where S_x is equal to the survival in waste concentration X and S_c is equal to the survival in the controls. If greater than 15% mortality occurred in the controls during the test, the test was considered invalid and no mortalities in the waste concentrations were calculated. To evaluate

TABLE 2. SUMMARY OF MEAN TOTAL RESIDUAL CHLORINE AND TOTAL AMMONIA LEVELS IN THE UNDILUTED EFFLUENTS DURING THE PERIOD OF FEBRUARY THROUGH AUGUST 1978.

Plant	Series	Effluent Type (a)	Mean TRC Residual (mg/l) (b)	Mean Total Ammonia (mg/l (b)
San Leandro	SL-1	UnCl	0.00 ± 0.00	13.9 ± 4.3
	SL-2	ECl	9.47 ± 2.42	12.6 ± 4.5
	SL-3	UnCl	0.00 ± 0.00	18.0 ± 3.6
	SL-4	ECl	5.93 ± 1.99	17.2 ± 2.4
San Pablo	SP-1	UnCl	0.00 ± 0.00	<0.1 ± 0.0
	SP-2	ECl	2.23 ± 0.74	<0.1 ± 0.0
	SP-3	DOHCl	2.15 ± 0.52	<0.1 ± 0.0
	SP-4	UnCl	0.00 ± 0.00	<1.0 ± 0.0
	SP-5	ECl	2.71 ± 0.47	<1.0 ± 0.0
	SP-6	DeCl	0.00 ± 0.00	<1.0 ± 0.0
	SP-7	UnCl	0.00 ± 0.00	<1.0 ± 0.0
	SP-8	ECl	2.44 ± 0.31	<1.0 ± 0.0
	SP-9	DOHCl	2.14 ± 0.31	<1.0 ± 0.0
Pinole	P-1	UnCl	0.00 ± 0.00	15.9 ± 4.1
	P-2	ECl	5.03 ± 2.32	14.9 ± 4.0
	P-3	DOHCl	3.01 ± 0.44	15.1 ± 4.4
	P-4	UnCl	0.00 ± 0.00	23.5 ± 7.8
	P-5	ECl	4.42 ± 2.16	22.8 ± 6.5
	P-6	DeCl	0.14 ± 0.52	23.9 ± 7.3
	P-7	UnCl	0.00 ± 0.00	19.2 ± 8.1
	P-8	ECl	8.00 ± 4.16	19.2 ± 8.3
	P-9	DOHCl	2.62 ± 0.36	19.4 ± 1.7
	P-10&11	ECl	4.39 ± 2.19	24.2 ± 11.7
South San Francisco	SSF-1&2	ECl	4.07 ± 2.02	64.5 ± 28.0

(a) ECl = Existing chlorinated effluent
UnCl = Existing unchlorinated effluent
DOHCl = Department of Health Pilot Plant Chlorinated effluent
DeCl = Existing dechlorinated effluent

(b) Mean ± standard deviation.

the toxicities of all effluents, we calculated a 96-h LC50 (the waste [%] or chlorine [mg/l TRC] concentration which produced 50% mortality to the test organisms within 96-h) or noted the mortality at the highest concentration tested for the test organisms.

Statistically significant ($P<.05$) differences among respective mean test results for the various optimized and existing chlorinated effluents were determined by subjecting the data to two-tailed t-tests [Sokal and Rohlf (17)]. Significant correlations ($P<.05$) between the toxicity of the chlorinated effluents and their chlorine residuals were calculated using linear regressions by the method of least squares (17).

RESULTS

Mean test chlorine residuals in the existing effluents varied considerably from a low at San Pablo of 2.23 mg/l TRC to a high of 9.57 mg/l TRC at San Leandro (Table 2). Mean chlorine residuals in the existing effluents at Pinole and South San Francisco were intermediate within this range.

In all cases, the optimized chlorination process lowered the mean chlorine residuals below those of the existing effluents (Table 2). Hence, it appears that the DOH Pilot plant optimized system uses chlorine more efficiently. During the comparative studies at San Pablo, both mean test chlorine residuals were lower in the optimized effluent, but only the second comparative study demonstrated a significant difference (Figure 4). Both comparative studies at Pinole demonstrated the optimized effluents to have significantly lower chlorine residuals than the existing effluents.

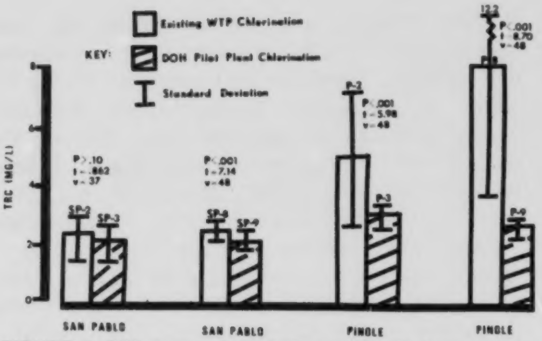

FIGURE 4. Differences in chlorine residuals between optimized and existing effluents

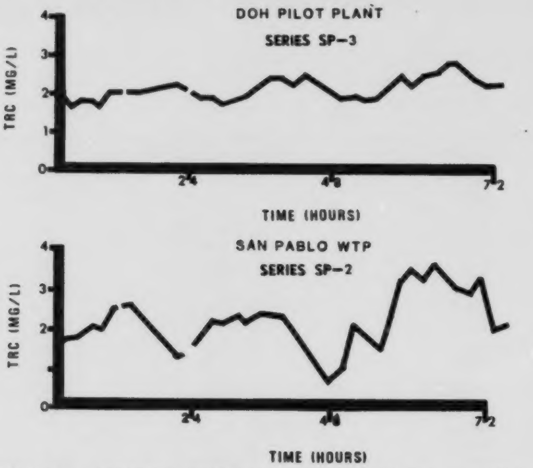

FIGURE 5. Chlorine residuals in optimized and existing effluents at San Pablo

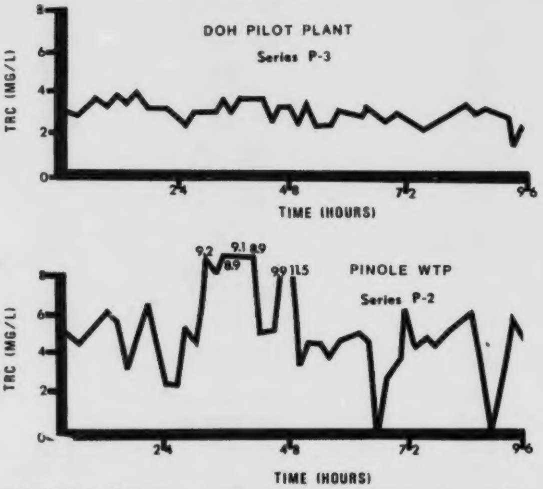

FIGURE 6. Chlorine residuals in optimized and existing effluents at Pinole

In addition to having comparatively lower chlorine residuals, the optimized effluents had typically less variable chlorine residuals (Table 2). This occurrence was apparent at both San Pablo (Figure 5) and at Pinole (Figures 6 and 7). The variability of chlorine residuals in both the optimized and existing chlorinated effluents during the second comparative test at San Pablo were similar (Figure 8). The existing effluents at San Leandro and South San Francisco also had variable chlorine residuals (Table 2).

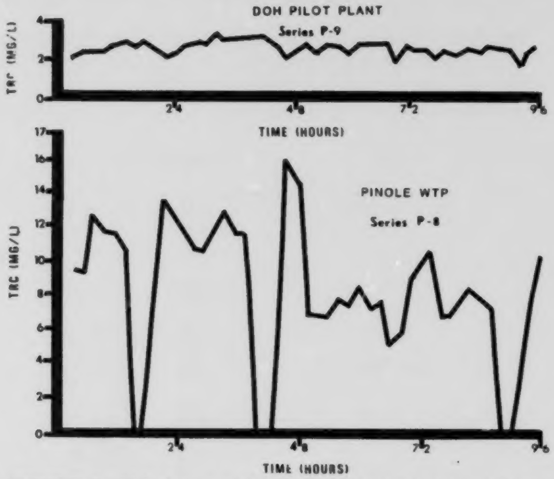

FIGURE 7. Chlorine residuals in optimized and existing effluents at Pinole.

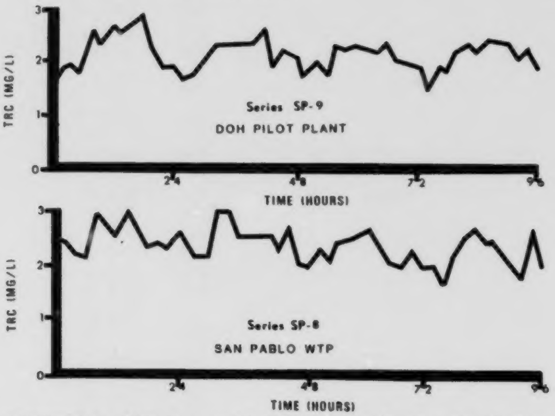

FIGURE 8. Chlorine residuals in optimized and existing effluents at San Pablo.

Acute Toxicity

There was no acute toxicity associated with either the existing unchlorinated or dechlorinated effluents at any of the plants investigated (Table 3).

Generally speaking, the toxicities of the optimized chlorinated effluents were less than those for the existing chlorinated effluents. One exception to this rule was during the first comparative study at San Pablo; the 96-h LC50's of the existing chlorinated effluents were greater than 50% while those of the optimized chlorinated effluents were 48%. During the second comparative study at San Pablo, the toxicity of the existing chlorinated effluent (96 – h LC50 $\bar{x} = 38\%$) was more than the optimized chlorinated effluent (96 – h LC50 $\bar{x} = 58\%$), but the difference between the two effluents was not significant. No comparison could be made between the two comparative effluents from the first study at Pinole because of excessive disease-related mortality which occurred in our test fish. During the second series of comparative testing at Pinole, the optimized chlorinated effluent (96 – h LC50 $\bar{x} = 15.5\%$) was significantly less toxic than the existing chlorinated effluent (96 – h LC50 $\bar{x} = 4.3\%$*)*.

Both fathead minnows and golden shiners were used as test organisms in the toxicity tests (Table 1). Golden shiners had to be substituted for fat-

TABLE 3. CONTINUAL-FLOW BIOASSAY RESULTS FOR UNCHLORINATED, CHLORINATED, AND DECHLORINATED EFFLUENTS AT WASTEWATER TREATMENT PLANTS IN CALIFORNIA FROM FEBRUARY THROUGH AUGUST 1978.

Plant	(a) Test Species	(b) Test Series		(c) Effluent Type	96-h LC50 (% Conc)	96-h LC50 (mg/l TRC)
San Leandro	FH	SL-1 (1)		UnCl	No toxicity (e)	
	FH	SL-1 (2)		UnCl	No toxicity	
	FH	SL-2 (1 + 2)	(d)	ECl	2.1	0.10
	FH	SL-3 (1)		UnCl	No toxicity	
	FH	SL-3 (2)		UnCl	No toxicity	
	FH	SL-4 (1)		ECl	3.5	0.13
	FY	SL-4 (2)		ECl	3.3	0.16
San Pablo	FH	SP-1 (1)		UnCl	No toxicity	
	FH	SP-1 (2)		UnCl	No toxicity	
	FH	SP-2 (1)		ECl	>50.0	>0.46
	FH	SP-2 (2)		ECl	>50.0	>0.52
	FH	SP-3 (1)		DOHCl	48.0	0.48
	FH	SP-3 (2)		DOHCl	48.0	0.65
	FH	SP-4 (1)		UnCl	No Toxicity	
	FH	SP-4 (2)		UnCl	No toxicity	
	FH	SP-5 (1)		ECl	43.0	0.53
	FH	SP-5 (2)		ECl	49.0	0.55
	FH	SP-6 (1)		DeCl	No toxicity	
	FH	SP-6 (2)		DeCl	No toxicity	
	FH	SP-7 (1)		UnCl	No toxicity	
	FH	SP-7 (2)		UnCl	No toxicity	
	FH	SP-8 (1)		ECl	39.0	0.59
	FH	SP-8 (2)		ECl	37.0	0.57
	FH	SP-9 (1)		DOHCl	71.0	0.90
	FH	SP-9 (2)		DOHCl	45.0	0.35
Pinole	FH	P-1 (1)		UnCl	ND (f)	
	FH	P-1 (2)		UnCl	ND (f)	
	FH	P-2 (1)		ECl	ND (f)	
	FH	P-2 (2)		ECl	ND (f)	
	FH	P-3 (1)		DOHCl	7.2	0.12
	FH	P-3 (2)		DOHCl	6.9	0.12
	GS	P-4 (1)		UnCl	No toxicity	
	GS	P-4 (2)		UnCl	No toxicity	
	GS	P-5 (1)		ECl	6.2	0.10
	GS	P-5 (2)		ECl	6.3	0.13
	GS	P-6 (1)		DeCl	No toxicity	
	GS	P-6 (2)		DeCl	No toxicity	
	GS	P-7 (1)		UnCl	No toxicity	
	GS	P-7 (2)		UnCl	No toxicity	
	GS	P-8 (1)		ECl	4.4	0.15
	GS	P-8 (2)		ECl	4.2	0.16
	GS	P-9 (1)		DOHCl	16.0	0.18
	GS	P-9 (2)		DOHCl	15.0	0.15

TABLE 3. (Continued).

Plant	Test Species [a]	Test Series [b]	Effluent Type [c]	96-h LC50 (% Conc)	96-h LC50 (mg/l TRC)
	GS	p-10 (1)	ECl	7.4	0.26
	GS	P-10 (2)	ECl	7.4	0.21
	FH	P-11 (1)	ECl	6.9	0.14
	FH	P-11 (2)	ECl	5.4	0.14
South San Francisco	FH	SSF-1 (1)	ECl	9.8	0.14
	FH	SSF-1 (2)	ECl	9.6	0.14
	GS	SSF-2 (1)	ECl	9.2	0.21
	GS	SSF-2 (2)	ECl	11.0	0.22

[a] FH = Fathead minnows *(Pimephales promelas)*
GS = Golden Shiners *(Notemigonus crysoleucas)*

[b] See Table 1 for test series dates; numbers in parentheses correspond to replicate tests.

[c] ECl = Existing chlorinated effluent
UnCl = Existing unchlorinated effluent
DOHCl = Department of Health Pilot Plant chlorinated effluent
DeCl = Existing dechlorinated effluent

[d] Results from the two replicates were averaged to estimate LC50.

[e] 100% survival in undiluted effluent.

[f] No LC50 determined due to excess mortality in controls.

head minnows when disease was present in our fathead minnow stock. We found the fathead minnow (96-h LC50 $\bar{x}=0.13$ mg/l TRC) to be significantly more sensitive to chlorine than the golden shiner (96 – h LC50 $\bar{x}=0.18$ mg/l TRC) by a factor of 140%.

We noticed a significant difference between the comparative toxicities of chlorine to fathead minnows at San Pablo and all of the other plants investigated (San Leandro, Pinole, and South San Francisco). Chlorine was found to be less toxic at San Pablo (96 – h LC50 $\bar{x}=0.58$ mg/l TRC) than at the other locations (96 – h LC50 $\bar{x}=0.13$ mg/l TRC) by a factor of 450%. The only measurable difference in the effluent quality between the two groups of plants was in their total ammonia content. San Pablo had ammonia levels consistently below 1.0 mg/l while all of the other locations had ammonia levels in excess of 10.0 mg/l (Table 2).

In effluents with total ammonia concentrations greater than 10.0 mg/l, we found significant correlations between the 96 – h LC50 levels and corresponding chlorine residuals of the undiluted effluents for both fathead minnows ($r=0.84$) and golden shiners ($r=0.86$)(Figure 9). The toxic reactions of chlorine to fathead minnows and golden shiners in those effluents appear to be similar since the slopes of the regression equations (-1.41 and -1.86, respectively) are similar. A nonsignificant correlation ($r=0.40$) was found between the two variables for fathead minnows at

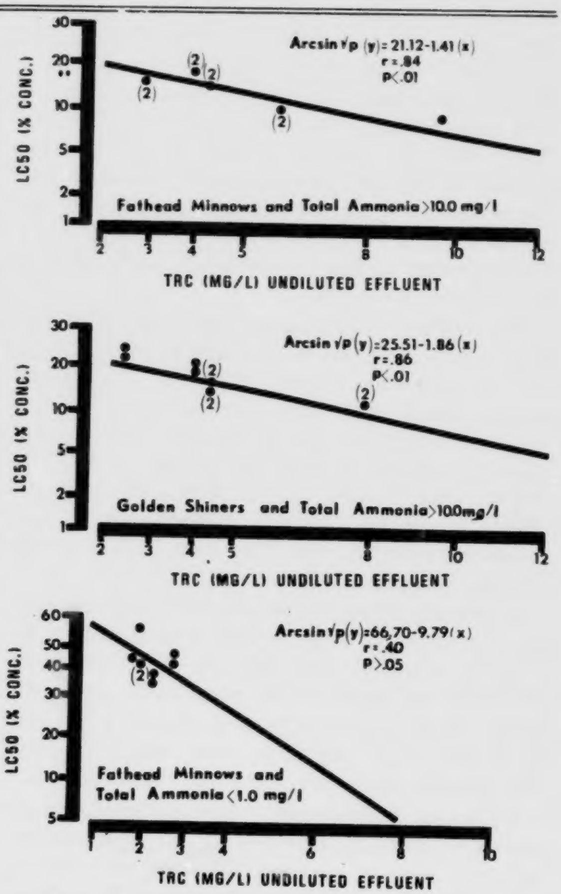

FIGURE 9. 96-HR LC50 estimates versus chlorine residuals

San Pablo where the undiluted effluents had less than 1.0 mg/l total ammonia.

DISCUSSION AND SUMMARY

No acute toxicity was found in any of the unchlorinated effluents. Dechlorination of the toxic chlorinated effluents removed all chlorine-induced toxicity. Hence, chlorine can be viewed as the single, most toxic constituent of most wastewater effluents [Martens and Servizi (11); Beeton, Kovacic, and Brooks (3)].

The optimized effluents were characteristically lower in residual chlorine than the existing effluents at both San Pablo and Pinole. Even though the optimized effluents at San Pablo had slightly lower mean chlorine residuals, the first comparative study showed the existing effluent to be lower in toxicity than the optimized effluent while the second comparative study showed the optimized effluent to be lower in toxicity than the existing effluent. Hence, chlorine optimization of the existing unchlorinated effluent at San Pablo did not produce toxicity less than that present in the existing chlorinated effluent. The San Pablo WTP may have been employing optimization design criteria into the chlorination of their effluents similar to that which was employed by the DOH pilot plant.

The optimized chlorinated effluents at Pinole were significantly lower in chlorine residual than the existing chlorinated effluents. During the second comparative study, the optimized effluent was significantly lower in chlorine and toxicity than the existing effluent by factors of 300 and 360%, respectively. In this case, chlorine optimization of the existing unchlorinated effluent produced a toxicity less than that present in the existing chlorinated effluent.

We found fathead minnows to be significantly more sensitive to chlorine than golden shiners by a factor of 140%. Our mean 96 – h LC50 values for both test species agree remarkably well with those values in the published literature [Arthur *et al* (2); Esvelt, Kaufman, and Selleck (8)] for ammoniated wastewater treatment plant effluents.

We found a significant difference in the sensitivities of fathead minnows to chlorine between nitrified and ammoniated effluents by a factor of 450%. The nitrified effluents at San Pablo had total ammonia concentrations less than 1.0 mg/l and a mean 96 – h LC50 of 0.58 mg/l TRC. Effluents from all other plants had total ammonia concentrations greater than 10.0 mg/l and a mean 96 – h LC50 of 0.13 mg/l TRC. The influence of ammonia on chlorine toxicity has been documented elsewhere. For example, Finlayson (9) found chlorine in ammoniated wastewater effluents to be more toxic to rainbow trout fry by a factor of 410% than in nitrified effluents containing less than 0.5 mg/l total ammonia.

A possible explanation for the observed difference in chlorine toxicity between these two types of effluents may in part be due to the relationships in the chemistry of chlorine and ammonia. At San Pablo, break-point chlorination was apparently accomplished because the chlorine dosages, which were in excess of twice the total ammonia concentrations, were above break-point [Collins and Selleck (7)]. Break-point chlorination was not apparently accomplished at any of the other plants investigated because the total ammonia concentrations were always higher than the chlorine residuals. At break-point chlorination ammonia is lacking; consequently, trichloramines, nitrous oxide, chlorine dioxide, and complex organochloramines may be produced from side-reactions between the chlorine and the waste. Conversely, when ammonia is present and abundant, mono- and dichloramines are primarily produced between the chlorine and waste.

Rosenberger (16) suspected mono- and dichloramines to be more toxic than the side-reaction products formed at break-point chlorination. The toxicities of trichloramines, nitrous oxide, chlorine dioxide, and complex organochloramines are not well known. However, all of the aforementioned chlorine residual species will partially titrate out as total residual chlorine in the amperometric titration [APHA (1)]. Hence, if one or more of these break-point chlorination products were present at San Pablo, and were either non-toxic or less toxic than mono- or dichloramine, and were included in the total chlorine residual, the result would be that the test fish would be able to withstand apparently higher chlorine residuals than the test fish at the other locations.

In summary, the most significant findings of this study are:

1. the optimized design criteria investigated in this study provided lower and more stable

chlorine residuals in wastewater than were provided by the existing facilities;

2. because the optimized effluents had lower chlorine residuals, they were typically lower in toxicity than existing effluents; and
3. nitrification of wastewaters prior to chlorination can reduce the toxicity of the resulting chlorine residuals.

In view of the latter finding, perhaps nitrification should be considered an important design criterion in chlorination systems.

ACKNOWLEDGEMENTS

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DISCUSSION

DR. DONALD JOHNSON: I am interested in the difference in the toxicity of the chloramines and particularly whether or not you had organic chloramines in the San Pablo effluent, which I understood had the lower toxicity . . . lower ammonia. Did you have any Kjeldahl nitrogen values for your wastewater?

MR. HANSEN: No. We had no Kjeldahl nitrogen measurements.

DR. JOHNSON: Were you really up at breakpoint? Normally the breakpoint ratio is 10 mg of chlorine per mg of ammonia. You said the ammonia concentrations were around 1 mg.

MR. HANSEN: I cannot recall the exact values but they were less than 1 mg/l, if we take all of the test into consideration. I think they got down to around 0.2 mg/l in some cases.

DR. JOHNSON: The values were 0.2 to 1 mg/l. What was your chlorine dosage?

MR. HANSEN: I do not know what the dosages were. We took the effluent that came from the optimized pilot plant, so the results I am reporting are what we measured in our laboratory.

DR. JOHNSON: Did you do any free chlorine titrations and compare with total chlorine?

MR. HANSEN: No, we just took the total.

DR. RIP RICE, *Jacobs Engineering:* Did I understand that all of your dechlorination was done with activated carbon?

MR. HANSEN: Yes, our dilution water was tap water which was dechlorinated with charcoal.

DR. RICE: Right. Was this powdered activated charcoal or was it granular?

MR. HANSEN: It was granular.

DR. RICE: Granular activated carbon?

MR. HANSEN: That is correct. We replaced that on each site.

DR. RICE: That is what I was going to ask. Did you have any data as to how long the dechlorination efficiencies . . .

MR. HANSEN: Two weeks. That is all we used it for and we replaced it 100 pounds at a time.

DR. RICE: And it was good for that length of time?

MR. HANSEN: Yes. No toxicity at all. We observed no mortality whatsoever in our controls and that is our guide.

MR. GREG SEEGERT, *WAPORA, Inc:* I had a question on the differences which you ascribe to the toxicity between the various fractions. You said in one case you were essentially dealing with solutions which you felt were primarily mono- and dichloramine and in the other case, since you said breakpoint chlorination was occurring, you were getting the various products such as trichloramine and some of the others. Is that correct?

MR. HANSEN: Yes, and that is mainly taken from the literature. That is just our attempt to explain why these differences occurred. We have not measured them ourselves.

MR. SEEGERT: It would seem to me that the more logical explanation would be the difference between the mono- and dichloramine versus the free chlorine fraction which, I think, is one of the things Don Johnson was essentially leading up to. In one case, if you have breakpoint chlorination occurring, your total residual in the breakpoint situation is going to be either hypochlorous acid or hypochlorite ion.

MR. HANSEN: Right.

MR. SEEGERT: In the other case you are going to be having the combination of primarily monochloramine with maybe a little dichloramine. It would be more logical to attribute any differences to that.

In support of that conclusion or the fact that there were differences in toxicity, you referred to a paper by Rosenberger. You said that he had reported that there were differences between the effects of monochloramine, dichloramine and trichloramine. I do not recall in that paper any tests on trichloramine. It was only free chlorine and monochloramine and dichloramine. He did not identify any trichloramine fraction.

MR. HANSEN: I believe that came out when he was discussing the various forms.

DR. JOHNSON: I might just say, I was thinking of organic chlorine rather free chlorine. I wonder if the free chlorine residual . . .

MR. HANSEN: I do not think we saw free chlorine. It disappeared pretty fast.

DR. JOHNSON: There is very little data on organic chloramine. That is the reason I asked about critical mass.

MR. SEEGERT: Well that is certainly true. There could be a sizeable fraction of more complex forms of chlorine, but there is likely a sizeable fraction of either hypochlorous acid or hypochlorite ion. Certainly in terms of killing bacteria and viruses, free chlorine is widely regarded as being more toxic.

DR. JOHNSON: Certainly it is the most toxic form, but I do not think that is what we are faced with, at least in the wastewater treatment plants. It is the chloramines which last longer, of course.

MR. VENOSA: Perhaps Endel could shed some light on the dosage of chlorine that was used in the pilot plant at that particular location.

MR SEPP: The chlorine dose at the San Pablo plant was 10-12 mg/l for the full scale plant and the dose in the pilot plant was 10-11 mg/l slightly lower.

Now these were the daytime dosages. We do not have any data on the night time dosages.

MR WHITE: I want to point out that the organic chloramines titrate in the forward titration with the dichloramines, so in wastewater treatment I would say it is impossible to be able to identify that something is dichloramine because there is probably no pure dichloramine.

Now, to learn about the effect of organic nitrogen in a breakpoint situation in both wastewater and tap water, Saunier did some research at the University of California that is in his thesis. I believe it was printed and excerpted in an AWWA paper that he gave. I do not know whether it has been in the journal or not but he tells about that.

6.
DESIGN OF A CHLORINATION SYSTEM FOR LAGOON DISINFECTION

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ABSTRACT

Chlorine disinfection of waste stabilization lagoon effluents has been and is being considered as a solution to bacterial removal prior to discharge to receiving waters. To evaluate the amenability of algae-laden lagoon effluent to chlorine disinfection, chlorination test facilities were constructed at the Logan, Utah, wastewater lagoons. An investigation was conducted at these facilities on primary and secondary, as well as filtered and unfiltered, lagoon effluents between August 1, 1975, and August 24, 1976. The filtered effluent was obtained by passing lagoon effluent through an intermittent sand filter prior to chlorination.

The results of this study indicate that, in all cases, adequate disinfection was obtained with combined chlorine residual within a contact period of 60 minutes or less. Filtered effluent was found to exert less chlorine demand than unfiltered effluent. It was also determined that temperature, sulfide, and total chemical oxygen demand influence the chlorine dose necessary to achieve a specified level of disinfection. Suspended solids and soluble chemical oxygen demand were found to be slightly altered as a result of chlorination.

A mathematical model was developed to represent the effects of chlorination of lagoon effluents. This model was used to predict the chlorine dosages necessary to achieve adequate disinfection for varying effluent characteristics. A series of design curves was constructed from the model for use in selecting the optimal chlorine dosages necessary for achieving prescribed levels of disinfection.

INTRODUCTION

Wastewater lagoon systems provide simple economic wastewater treatment for approximately 5,000 communities throughout the United States. Historically, hydraulic detention times within many of these lagoon systems have been sufficient to provide adequate total and fecal coliform reduction. However, as a result of more stringent state and federal discharge standards, many proposed and existing lagoon systems may not produce satisfactory effluent coliform concentrations. Therefore, disinfection of lagoon effluents is becoming common practice.

Although various disinfection processes are available for application to lagoons, chlorination is practiced in most installations. However, there is evidence [Burkhead and O'Brien, 1973 (3); Echelberger, *et al.*, 1971 (8); and Hom, 1972 (11)] that chlorination of lagoon effluents increases the biochemical oxygen demand (BOD_5) and soluble chemical oxygen demand of the effluent. Thus, there is serious concern about the proper chlorine dose and contact time required for satisfactory chlorination of lagoon effluents.

The paper presents a series of design curves based on a mathematical model developed by Johnson, *et al.*, 1978 (12). The model was developed with data obtained from operating a field scale waste stabilization lagoon facility for twelve months to evaluate lagoon chlorination practices under varying seasonal conditions. This evaluation included determination of the chlorine residual, concentration necessary to reduce bacterial population to acceptable levels, and the effects of temperature, suspended solids, ammonia, chemical oxygen demand, and sulfide on chlorination practices.

PROCEDURES

The Logan, Utah, wastewater stabilization lagoons were selected as the site for this study. Because of the relatively high bacteriological quality of the final lagoon effluent, the facilities were constructed with capabilities of treating either primary or secondary lagoon effluent. Four systems of identically designed chlorine mixing and contact tanks, each capable of treating 50,000 gallons per day, were constructed. Using recommendations presented by Collins, *et al.*, 1971 (5), Kothandaraman and Evans, 1972 (13) and 1974 (14) and Marske and Boyle, 1973 (16), the chlorination systems were constructed to provide rapid initial mixing followed by chlorine contact in plug flow reactors. A serpentine flow configuration, having a length to width ratio of 25:1, coupled with inlet and outlet baffles, was used to enhance plug flow hydraulic performance. The chlorine mixing and contact tanks are illustrated in Figure 1. Dye studies similar to those conducted by Deaner [undated] (6) were used to determine average detention times for each contact tank. The theoretical detention time for each tank was 60 minutes, while the actual detention time for the four tanks averaged 49.6 minutes.

Three of the four chlorination systems were used for directly treating primary and secondary lagoon effluent. The effluent treated in the fourth system was filtered through an intermittent sand filter prior to chlorination. Filtered lagoon effluent was also used as the solution water for all four chlorination systems. Appropriate quantities of chlorine gas were mixed with solution water by use of vacuum operated diffusers prior to introducing solution lines into chlorine mixing tanks and exposing the main flow of lagoon effluent to chlorine. Schematic diagrams to illustrate the chlorination operation are presented in Figures 2 and 3.

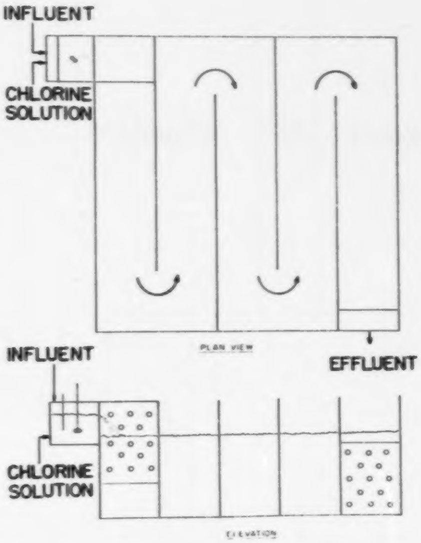

Figure 1. Chlorine mixing and contact tanks.

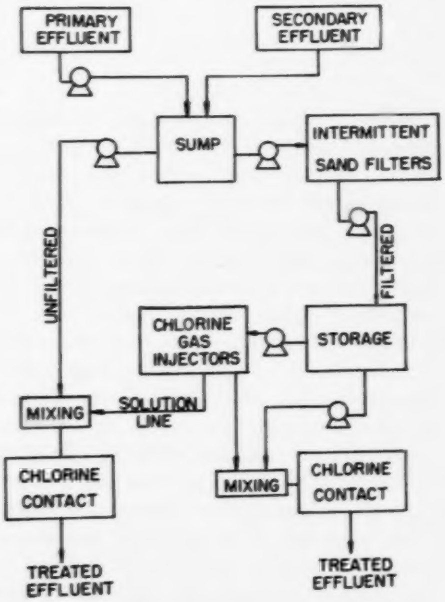

Figure 2. Schematic flow diagram of experimental chlorination facility.

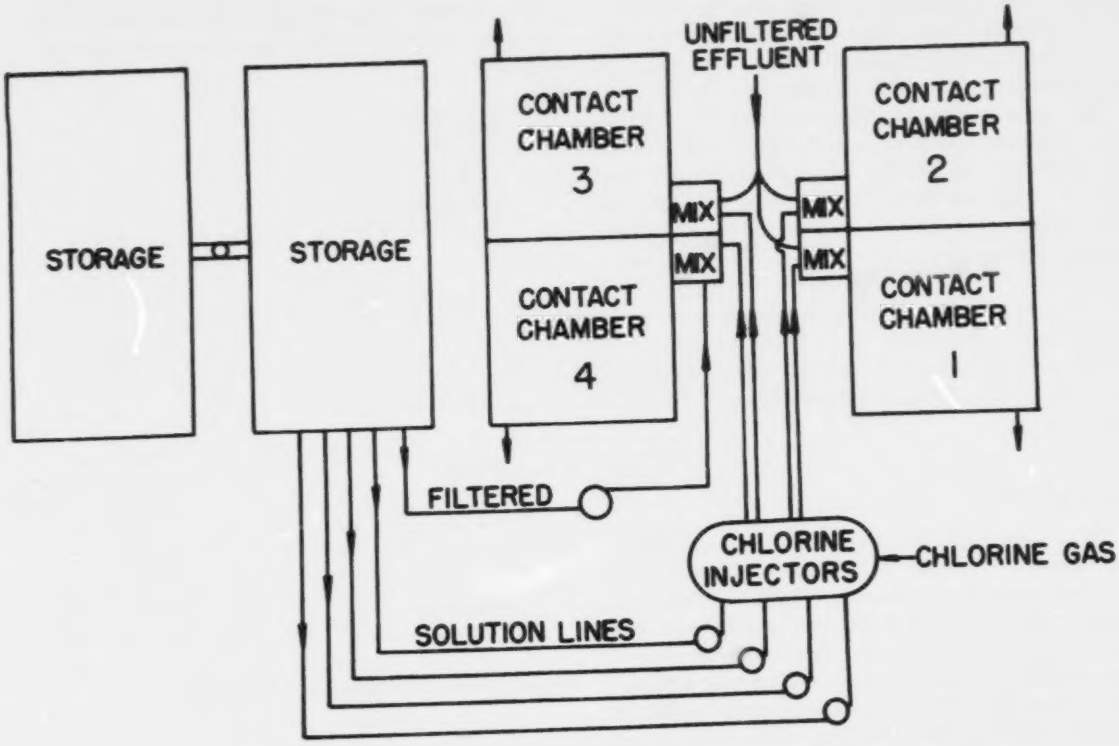

Figure 3. Chlorination facilities.

Chlorination of lagoon effluent began on August 6, 1975, and continued until August 24, 1976. Samples were collected at least twice a week throughout the study period except between December, 1975, and February, 1976. No samples were collected during that period because of pipeline freeze-up. Samples were collected just prior to chlorination as well as at points corresponding to residence times of 17.5, 35.0, and 49.6 minutes in each contact tank. Chlorine doses were varied between 0.25 and 30.0 mg/l. In addition to chlorinated samples, other samples were collected from the influent and effluent of the lagoon system and from the effluent from each cell in the system. This was done to characterize the performance of the lagoon system and to assist in determining how to adjust chlorination practices to compensate for seasonal fluctuations in lagoon performance.

The chlorinated samples were analyzed bacteriologically for MPN total and fecal coliforms (TC and FC). Five tubes were used for each dilution. Membrane filter total and fecal coliform determinations were also conducted on all unchlorinated samples. Additional water quality analyses included ammonia (NH_3-N), biochemical oxygen demand (BOD_5), dissolved oxygen (DO), total and soluble chemical oxygen demand (TCOD and SCOD), sulfide ($S^=$), suspended solids (SS), volatile suspended solids (VSS), pH, temperature, and turbidity. Free and combined chlorine residuals (FCl and CCl) were also measured for all chlorinated samples using the amperometric titration method. With the exception of the sulfide, all samples were collected and analyzed using recommended procedures outlined in APHA Standard Methods, 1971 (1). Sulfide was analyzed using a method described by Orion [undated] (17).

In addition to the field study as described, laboratory studies were also conducted. These studies were performed to assist in describing relationships between chlorine and other wastewater constituents Johnson, *et al.*, 1978 (12).

RESULTS AND DISCUSSION

Lagoon Disinfection

General

To determine disinfection efficiency, with respect to total and fecal coliforms, as a function of total chlorine residual, the data were fitted with a linear regression equation, which expresses the logarithm of the fraction of coliform remaining as a function of total chlorine residual. The intercept values for these regression equations range from -0.3 to -1.3, indicating that a zero total chlorine residual concentration will produce a coliform reduction of from 50 percent to 95 percent. A possible explanation for this anomaly is that chlorine combines with ammonia, organic materials, sulfides, and other compounds and thus dissipates leaving no measurable chlorine residual. However, at some point in time (perhaps only an instant after addition) the chlorine is also in contact with the bacteria in the water and available for disinfection, resulting in decreases in coliform concentrations indicated by the intercept values. The intercept values may also be due to the statistical confidence intervals associated with the MPN values.

In an attempt to relate the results of this study with those reported in the literature, a similar regression analysis as discussed above was performed using a forced zero intercept. The results of this analysis are illustrated in Figures 4, 5, 6, and 7. The correlation coefficients were significant at the 5 percent level for all reported regression equations. Because of this high degree of statistical significance, the forced zero intercept regression analysis was used in discussing the results. These results are expressed as the $\log_{10} N/N_0$ (in which N = the number of organisms per 100 ml after chlorination, and N_0 = the original number of organisms per 100 ml), or the logarithm of the fraction remaining after chlorination versus varying concentrations of total chlorine residual.

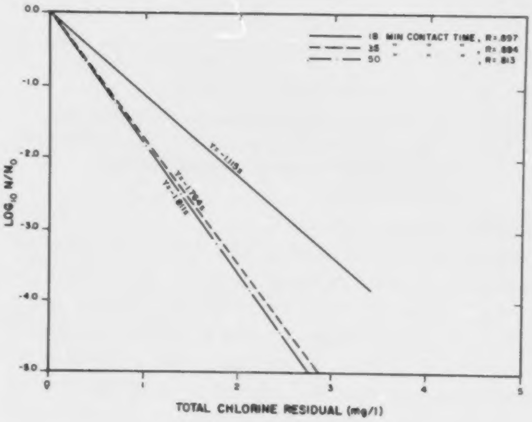

Figure 5. Summary of fecal coliform removal efficiency in filtered lagoon effluent as a function of total chlorine residual at various chlorine contact times.

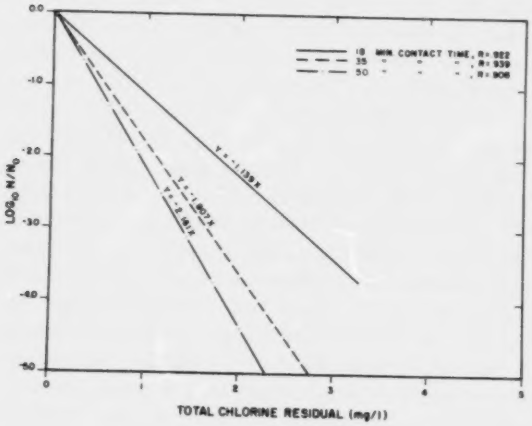

Figure 4. Summary of total coliform removal efficiency in filtered lagoon effluent as a function of total chlorine residual at various chlorine contact times.

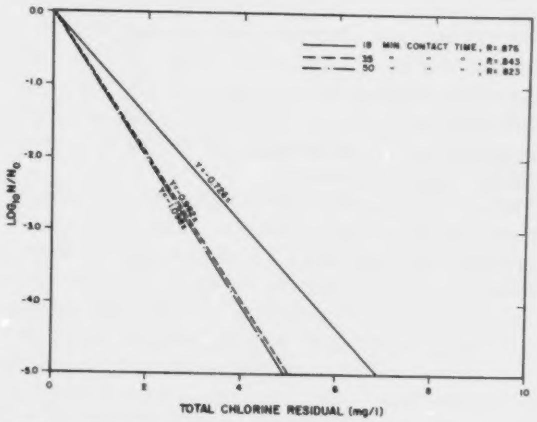

Figure 6. Summary of total coliform removal efficiency, using unfiltered lagoon effluent, as a function of total chlorine residual at various chlorine contact times.

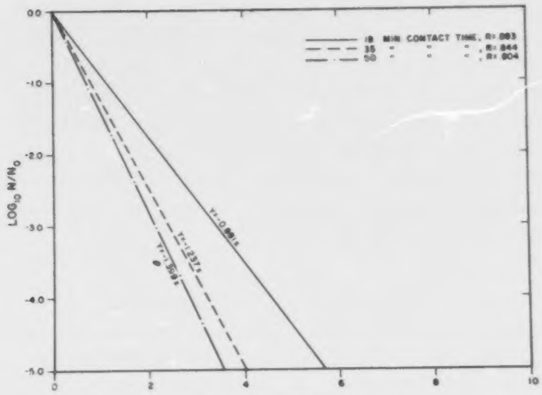

Figure 7. Summary of total coliform removal efficiency, using unfiltered lagoon effluent, as a function of total chlorine residual at various chlorine contact times.

Filtered Lagoon Effluent

Total coliforms. The effect of total chlorine residual on total coliform numbers after chlorine contact times of 18, 35, and 50 minutes using filtered lagoon effluent is reported in Figure 4. Analysis of Figure 4 indicates that the rate of total coliform removal increases with increasing total chlorine residual. This result is in agreement with reports in the literature, [Butterfield, 1948 (2); Chambers, 1971 (4); Green and Stumpf, 1944 (9); Hom, 1970 (10); Kott, 1971 (15); and White, 1972 (18)]. Results from Figure 4 indicate that a total coliform organism reduction of 99.9 percent in filtered lagoon effluent can be expected with a total chlorine residual concentration of 2.7 mg/l after 18 minutes chlorine contact time. The 99.9 percent level of reduction was chosen for discussion because at this removal efficiency the data and subsequent regression lines are well developed and interpretation of results is accomplished with less inference.

By using a statistical test which employs sums of squares, sums of products, degrees of freedom, sample regression coefficients, and regression equation slopes, it was determined that the slopes of the regression equations for the 35 and 50 minute chlorine contact times reported in Figure 4 were not significantly different from one another and could be regarded as having the same slope. This statistical procedure is used in determining whether confidence intervals for two regression lines overlap. If an overlap does occur, it is an indication that the regression equations (regression lines) are not statistically different. Statistical results of this kind further suggest that the two regression lines describe a range in which a single regression line would be found. Therefore, if the data for both the 35 and 50 minute chlorine contact times were analyzed together and fitted with a regression equation, the line described by this equation would fall somewhere between 35 and 50 minute chlorine contact time regression lines shown in Figure 4. However, this approach was not used because it was apparent that to group two different operational time period data would be statistically incorrect. Because these two regression lines are not significantly different and because grouping data is invalid, interpolation between the 35 and 50 minute chlorine contact time regression lines in Figure 4 indicate that a total chlorine residual of 1.5 mg/l is required to produce a 99.9 percent total coliform reduction at chlorine contact times between 35 and 50 minutes. This concentration is contrasted with the 2.7 mg/l total chlorine residual needed for the same level of reduction at the 18 minute chlorine contact times, which is consistent with earlier reports [Butterfield, 1948 (2); Chamber, 1971 (4); and White, 1972 (18)].

The above experimental and statistical results imply that an increase in chlorine contact time from 18 to 35 minutes will require 1.2 mg/l or 44 percent less total chlorine residual to obtain the same level of total coliform destruction. However, at chlorine contact times between 35 and 50 minutes, there is no statistically significant difference in total chlorine residual required to produce a 99.9 percent reduction in total coliform concentration. A possible explanation for this effect occurring consistently at the 35 and 50 minute chlorine contact time is that coliform concentrations are reduced to such low levels within the 35 minute chlorine contact time that further reductions with increasing time are not statistically measurable.

Fecal coliforms. The effects of total chlorine residual on fecal coliform bacteria in filtered lagoon effluent are illustrated in Figure 5 at 18, 35, and 50 minutes of chlorine contact time. This figure depicts a trend in reduction of fecal coliform levels with increasing total chlorine residual concentration and chlorine contact time similar to the trend indicated by Figure 4 for total coliform reduction. The 35 and 50 minute chlorine contact time regression lines for Figure 5 were also found to have statistically similar slopes. An average of 1.7 mg/l total chlorine residual after 35 to 50 minutes contact time is needed to effect a three log fecal coliform reduction. At the 18 minute chlorine contact time, 2.7 mg/l total chlorine residual were required to reduce fecal coliform bacteria to this same level. This is a difference of 1.0 mg/l or 37 percent less total chlorine residual. This suggests longer chlorine contact times will produce the same level of fecal coliform removal as higher total chlorine residual concentrations. Again, these results are similar to earlier published reports: Chambers, 1971 (4); Hom, 1970 (10); and White, 1972 (18).

Unfiltered Lagoon Effluent

Total coliform. Total coliform reduction versus residual chlorine at three different contact times (18, 35, and 50 minutes) is illustrated in Figure 6. Figure 6 indicates, again, that increasing amounts of total chlorine residual and chlorine contact time will increase the rate of reduction of total coliform numbers. This indicates that a 99.9 percent reduction of total coliform bacteria can be achieved with 18 minutes of chlorine contact at total chlorine residual concentrations of 4.2 mg/l. The regression lines in this figure illustrating the 35 and 50 minute chlorine contact times are statistically the same, for reasons discussed earlier in connection with filtered lagoon effluent, and suggest that an average of 3.0 mg/l total chlorine residual is required to reduce total coliform concentrations by 99.9 percent at these chlorine contact times. This is a 29 percent, or 1.2 mg/l, reduction of total chlorine residual for the 35 to 50 minute chlorine contact times over that for the 18 minute chlorine contact time.

Fecal coliform. The effect of total chlorine residual on fecal coliform reduction in unfiltered lagoon effluent is shown by Figure 7 for chlorine contact times of 18, 35, and 50 minutes, respectively. Figure 7 indicates reduced fecal coliform concentrations with increased chlorine contact time and total chlorine residual. A 3.4 mg/l total chlorine residual with an 18 minute chlorine contact time and an average of 2.3 mg/l of total chlorine residual for 35 and 50 minutes chlorine contact times are suggested by this figure to reduce fecal coliform levels by 99.9 percent. This means 32 percent less total chlorine residual is required at the longer contact times than for that at 18 minutes chlorine contact to produce the same level of reduction.

Results further indicate that the 99.9 percent fecal coliform reduction level is achieved at lower concentrations of total chlorine residual than for total coliform reduction in unfiltered lagoon effluent (an average of 5.7 mg/l total chlorine residual for fecal coliform reduction compared to 7.2 mg/l total chlorine residual for total coliform reduction, or 21 percent less). Fecal coliform bacteria may be less resistant to chlorine in unfiltered lagoon effluent. The difference between wastewater characteristics found in unfiltered lagoon effluent and those in filtered effluent in combination with chlorine may cause the fecal coliform to die off at a greater rate in the unfiltered lagoon effluent. This may help to explain why total chlorine residual concentrations vary in effectiveness between total and fecal coliform reduction with unfiltered lagoon effluent and not with filtered lagoon effluent.

Summary

Results indicate that increasing total chlorine residual will produce increased total and fecal coliform reduction for both filtered and unfiltered lagoon effluent. Results also suggest that statistically significant reductions in coliform concentration can be accomplished at the same residual chlorine level with increasing chlorine contact times.

Indications are that less total chlorine residual is required for disinfection of filtered lagoon effluent over that of unfiltered lagoon effluent. An average of 3.6 mg/l total chlorine residual is required for a 99.9 percent reduction of total coliform numbers in unfiltered lagoon effluent, a 42 percent lower chlorine residual requirement. In addition, 23 percent less total chlorine residual (2.2 mg/l) is required to achieve a 99.9 percent reduction of fecal coliform numbers in filtered lagoon effluent, as compared with unfiltered lagoon effluent (2.85 mg/l).

There is also evidence that fecal coliform bacteria in unfiltered lagoon effluent are reduced to the 99.9 percent level with smaller concentrations of total chlorine residual than are the total coliform bacteria.

MODEL DEVELOPMENT

General

The mathematical model developed by Johnson, *et al.*, 1978 (12) predicts the appropriate combination of chlorine dose, contact time, and chlorine residual for adequate disinfection of a specific waste stabilization lagoon effluent. The model accounts for the interactions and temperature dependence of chlorine and sulfide, nitrogen compounds, chemical oxygen demand, suspended solids, chlorine demand, and disinfection (i.e., coliform reduction). Field and laboratory data were evaluated to develop these relationships and the coefficients employed in the model.

Model development

The model incorporates the equations shown in Table 1. In developing the mathematical model, steady state representations of Equations 1 to 6 were used to determine the chlorine consumed by sulfide, the sulfide remaining after chlorination, and reactions with ammonia and organic nitrogen. The dynamic portion

TABLE 1. SUMMARY OF EQUATIONS EMPLOYED TO DEVELOP DESIGN CURVES.

Function	Equation Number	Equation*	Comments
To predict the amount of sulfide remaining after chlorination	1	$S = S_L + (S_0 - S_L)\, e^{-.141(x)}$	form = exponential decay r = 0.674 significance level = 95%
To describe the interreactions between chlorine and nitrogen forms	2 3 4 5 6	$Cl_2 + H_2O \rightleftharpoons HOCl + H^+ + Cl^-$ $HOCl \rightleftharpoons OCl^- + H^+$ $HOCl + NH_3 \rightleftharpoons NH_2Cl + H_2O$ $HOCl + NH_2Cl \rightleftharpoons NHCl_2 + H_2O$ $HOCl + NHCl_2 \rightleftharpoons NCl_3 + H_2O$	Assumption is that reactions occur rapidly enough to be instantaneous. Thus, steady state is assumed.
To describe the effects of chlorine on suspended solids and COD	7	$\frac{d(SCOD)}{dt} = 7.24 \times 10^{-4}\,(FCl)(TCOD - SCOD)$	Rate constant determined from regression analysis (l/mg-min)
To describe effects of chlorine on suspended solids	8	$\frac{d(SS)}{dt} + (-3.0 \times 10^{-4})(SS)(FCl + CCl)$	Rate constant determined from regression analysis (l/mg-min)
To determine changes in free chlorine (FCl) and combined chlorine (CCl)	9 10	$\frac{d(FCl)}{dt} = (-.20)\ \frac{FCl}{FCl_0}^{7.4}\ (TCOD)$ $\frac{d(CCl)}{dt} = (-.10)\ \frac{CCl}{CCl_0}^{7.4}\ (TCOD)$	Values for rate and stoichiometric constants determined in model calibration process.
To determine changes in total coliform concentrations (TC and fecal coliform concentrations (FC)	11 12	$\frac{d(TC)}{dt} = (-.055)(TC)^{1.1}(CCl)^{1.35} + (-.20)(TC)^{1.1}(FCl)^{1.30}$ $\frac{d(FC)}{dt} = (-.085)(FC)^{1.08}(CCl)^{1.35} + (-.35)(FC)^{1.06}(FCl)^{1.30}$	Values for rate and stoichiometric constants determined from regression analysis and model calibration process.
To account for temperature effects	13	$k_T = k_{20}\ \beta^{(T-20)}$	β = 1.15 for chlorine demand β = 1.03 for disinfection

*See LIST OF SYMBOLS.

of the model, as represented by Equations 7 to 12 to describe changes in soluble COD, SS, free and combined chlorine residual, and total and fecal coliforms, was solved using a second order Runge-Kutta solution technique. Temperature effects were accounted for by using equation 13.

The model was calibrated by first selecting approximately 5 percent of the total data (1804 total data points) representing extremes in effluent characteristics as the calibration data. The model was then applied to these data and coefficients adjusted by trial and error until a maximum value of the correlation coefficient, r, was achieved. A sensitivity analysis was performed to determine those coefficients most sensitive in producing changes in the model results. It was found that those stoichiometric constants associated with total and fecal coliform in the disinfection equations were the most sensitive.

The model was verified by comparing the predicted values produced by the model with all the observed values of the field data. Predicted combined chlorine residuals were found to be correlated to 65 percent of the observed data (1804 total data points) at the 95 percent confidence level, while 81 percent of predicted total and fecal coliform values compared with observed values at the same confidence level.

DESIGN CURVES

The model was then used to construct a series of design curves for selecting chlorine doses and contact times for achieving desired levels of disinfection. An example may best illustrate how these design curves are applied. Assume that a particular lagoon effluent is characterized as having a fecal coliform concentration of 10,000/100 ml, 0 mg/l sulfide, 20 mg/l TCOD, and a temperature of 5°C. If it is necessary to reduce the fecal coliform counts to 100/100 ml, a combined chlorine residual sufficient to produce a 99 percent bacterial reduction must be obtained. If an existing chlorine contact chamber has an average residence time of 30 minutes, the required chlorine residual is obtained from Figure 8. A 99 percent bacterial reduction corresponds to log (N_0/N) equal to 2.0. For a contact period of 30 minutes, a combined chlorine residual of between 1.0 and 1.5 mg/l is required to produce that level of fecal coliform reduction. Upon interpolation, the actual chlorine residual is determined to be 1.3 mg/l. This is indicated by point ① in Figure 8.

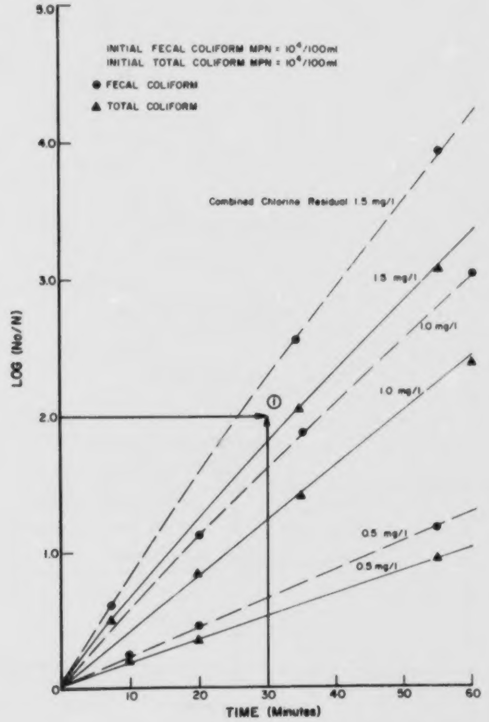

Figure 8. Combined chlorine residual at 5°C for coliform = 10^4/100 ml.

Going to Figure 9, it is determined that if a chlorine dose produces a residual of 1.30 mg/l at 5°C, the same dose would produce a residual of 0.95 mg/l at 20°C. This is because of the faster rate of reaction between TCOD and chlorine at the higher temperature. This is indicated by point ② in Figure 9. For an equivalent chlorine residual of 0.95 mg/l at 20°C and 20 mg/l TCOD, it is determined from Figure 10 that the same chlorine dose would produce a residual of 0.80 mg/l if the TCOD were 60 mg/l. This is because higher concentrations of TCOD increase the rate of exertion of chlorine demand. Point ③ in Figure 10 corresponds to this residual. The chlorine dose required to produce an equivalent residual of 0.80 mg/l at 20°C and 60 mg/l TCOD is determined from Figure 11. For a chlorine contact period of 30 minutes, a chlorine dose of 2.15 mg/l is necessary to produce the desired com-

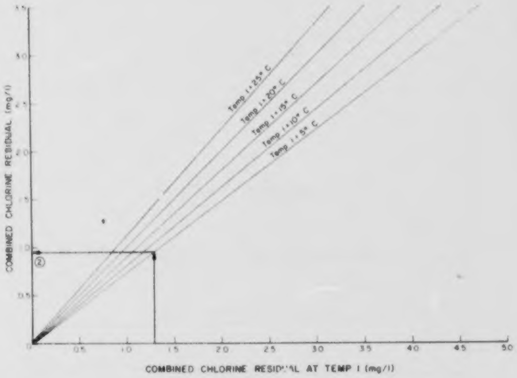

Figure 9. Conversion of combined chlorine residual at Temp 1 to equivalent residual at 20°C.

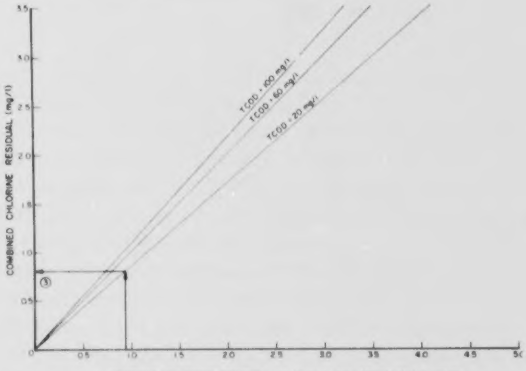

Figure 10. Conversion of combined chlorine residual at TCOD 1 and 20°C to equivalent residual at 20°C and TCOD = 60 mg/l.

bined residual as indicated by point ④ on Figure 11. This dose will produce a reduction of fecal coliform from 10,000/100 ml to 100/100 ml within 30 minutes at 5°C and with 20 mg/l TCOD.

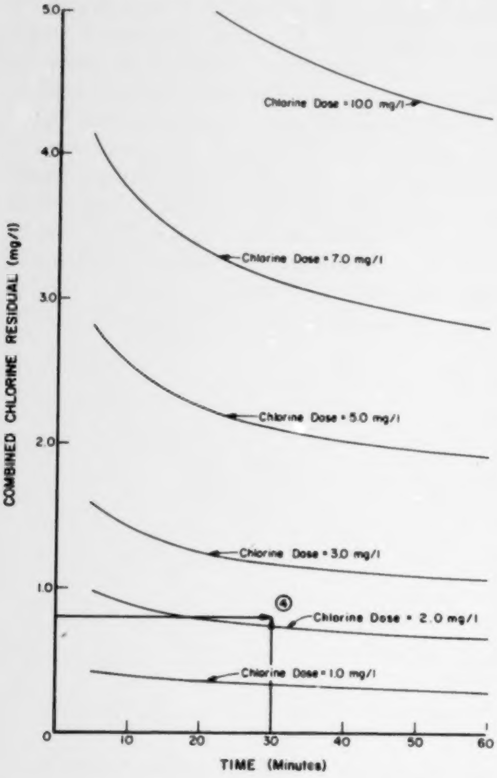

Figure 11. Determination of chlorine dose required for equivalent combined residuals at TCOD = 60 mg/l and Temp = 20°C.

If, in the previous example, the initial sulfude concentration was 1.0 mg/l instead of 0 mg/l, it would be necessary to go directly from Figure 8 to Figure 12. Here, chlorine residual of 1.30 mg/l at a TCOD of 20 mg/l and a temperature of 5°C is converted to an equivalent chlorine residual of 1.10 mg/l for a TCOD of 60 mg/l. This is represented by point ⑤ in Figure 12. Going to Figure 13, which corresponds to an initial sulfide concentration of 1.0 mg/l, it is determined that a chlorine dose of 6.65 mg/l is necessary to produce an equivalent chlorine residual of 1.1 mg/l after a contact period of 30 minutes. Point ⑥ on Figure 13 corresponds to this dose. The sulfide remaining after chlorination is determined to be 0.44 mg/l from Figure 14 as indicated by point ⑦.

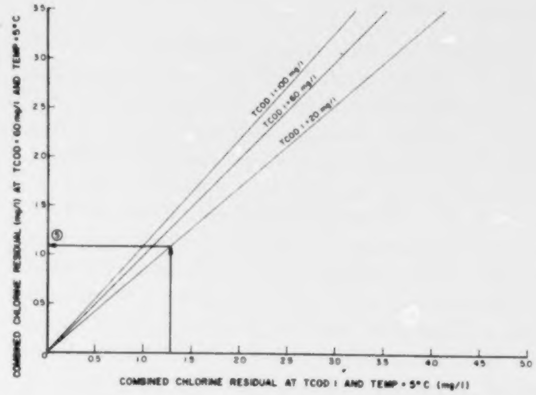

Figure 12. Conversion of combined residual chlorine at 5°C and TCOD 1 to equivalent residual at 5°C and TCOD = 60 mg/l.

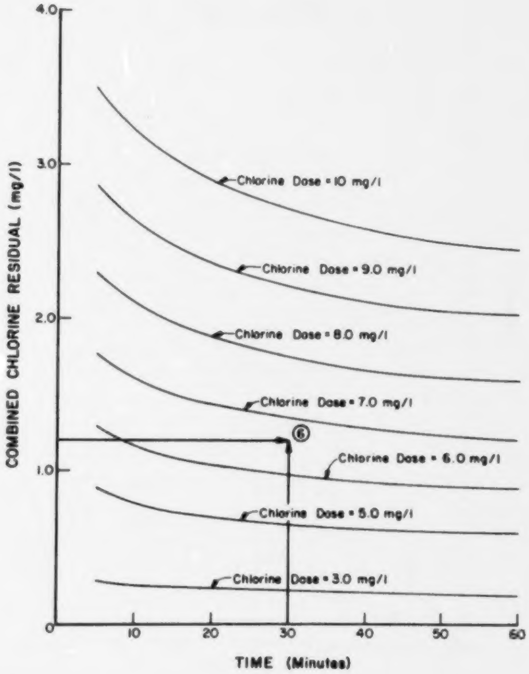

Figure 13. Determination of chlorine dose required when $S^{=}$ = 1.0 mg/l, TCOD = 60 mg/l, and Temp = 5°C.

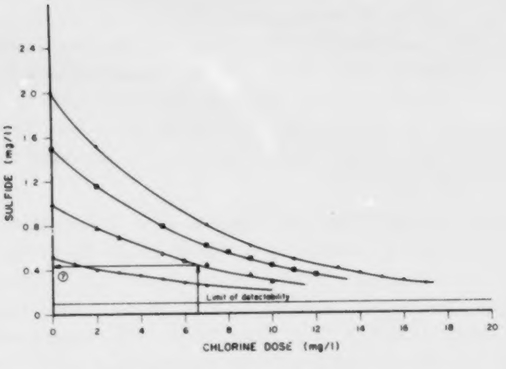

Figure 14. Sulfide reduction as a function of chlorine dose.

SUMMARY

A disinfection model was developed, verified, and used to produce design curves which are applicable to most lagoon systems. The model reflects the following variations and trends.

It was determined that disinfection of waste stabilization lagoon effluent can generally be achieved with relatively low doses of chlorine and in contact times of less than 50 minutes. The chlorine demand was found to be less than reported in other literature during most of the year. Generally it was found that the chlorine demand was about 50 percent of the applied dose during all times of the year except when hydrogen sulfide was produced. During that period, the chlorine demand was found to be as high as 85 percent. Combined chlorine residuals of between 0.5 and 1.0 mg/l were found to be adequate in reducing fecal coliforms below the discharge standard of 200/100 ml. This residual is produced by a chlorine dose of between 2-3 mg/l, except during periods of hydrogen sulfide production when a dose of 7-8 mg/l is required.

Chlorination of these algae laden waters was accompanied by very few and minor adverse effects. Soluble COD was observed to increase in the presence of free chlorine residual. Increases in turbidity and reductions of SS were also observed for high chlorine doses. However, it was rarely necessary to chlorinate at high enough doses for these responses to have any major repercussions. Breakpoint chlorination was observed to be of minimal importance in providing adequate disinfection. Filtering of lagoon effluent through intermittent sand filters prior to chlorination was found to reduce chlorine demand and enhance disinfection efficiency.

The disinfection model was used to develop a series of design curves (seven graphs) which may be used to determine the chlorine doses, contact time, and chlorine residual required for a given level of disinfection. The use of the curves is illustrated by two separate examples.

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

β = Temperature coefficient
CCl = Combined chlorine residual, mg/l
CCl_0 = Initial combined chlorine residual, mg/l
FC = Fecal coliform, N_0/100 ml
FCl = Free chlorine residual, mg/l
FCl_0 = Initial free chlorine, mg/l
S = Sulfide
S_L = Lower limit of sulfide detection (0.1 mg/l)
S_0 = Initial sulfide concentration, mg/l
SCOD = Soluble COD, mg/l
SS = Suspended solids, mg/l
TCOD = Total COD, mg/l
TC = Total coliform, N_0/100 ml
X = Chlorine dose, mg/l

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DISCUSSION

DR. KARL LONGLEY, *U.S. Army:* I found it interesting that you based your nomographs on residual as a function of dose for 60 mg/l TOC. In work that I did with the University of Texas at the Rilling Road Plant at San Antonio, we were not able to find good correlation. We were not able to accurately predict the residual as a function of the dose based on TOC. It was interesting to see that you were able to do it for your particular system.

I would like to reemphasize what you said at the end, and that is the fact that you did a tremendous amount of work here to arrive at your model. The danger will be of course, particularly when this report becomes available to consulting engineering firms and whomever, that there will be a tendency to take these numbers without checking them on other systems and seeing if the factors hold up. It is a very dangerous practice but, unfortunately, it is followed by too many consultants.

DR. REYNOLDS: In response to your first comment, it had been reported in the literature that chlorination of lagoon effluent seriously affected soluble COD particularly or soluble BOD, thereby releasing dissolved organics into the effluent. We found that only occurred in our practice, when excessive amounts of chlorine were added to the lagoon effluent, specifically, when breakpoint was approached. As long as the chlorine residual was less than 5 mg/l, and it was not above breakpoint conditions, we did not see the disruption of the algal cell wall and the release of dissolved organics. That may explain the discrepancy between what you have observed in Texas and what we observed.

We did not observe release of dissolved organics in the effluent due to chlorination, unless massive doses of chlorine were used. We studied doses from 0.2 mg/l up to as high as 30 mg/l.

I agree with your comment about application of the model.

MR. RON SOLTIS, *Washington Suburban Sanitary Commission:* Was the floating material on top of the lagoon allowed to be discharged with the effluent into your filters? I noticed in the pictures what looked like green material on the top.

DR. REYNOLDS: Yes. The first picture we showed illustrated that there was some floating algal mass on top. The discharge at that point was submerged. There was no attempt to prevent that from going over if it did occur. So, sometimes it did and sometimes it did not depending on how things went. Generally, we did not have mats of algae floating in the chlorine contact chambers.

MR. SOLTIS: Did the floating material ever cover the entire lagoon?

DR. REYNOLDS: No.

MR. WHITE: You flashed one of those slides too fast for me. A couple of questions. Was this primary effluent discharging to lagoons or was it raw sewage discharging to lagoons?

DR. REYNOLDS: The lagoon system itself treats raw wastewater without any prior treatment. There is no comminution. There is a bar screen at the head of the lagoon system but no primary treatment and no preliminary treatment.

MR. WHITE: What was the calculated length of the residence time in the lagoon?

DR. REYNOLDS: The lagoon system was designed for a wintertime capacity of 180 days and a summertime capacity of 120 days. There have been no concrete studies to indicate what the actual hydraulic residence time is. We did calculate on one of **the ponds, which had a theoretical residence time of 9 days, that it was actually 9 hours, so we would assume that in the primary cell it was substantially less than the 90 days that it was designed for. How much less we do not know.**

MR. WHITE: How deep were they?

DR. REYNOLDS: They averaged in depth from about 5 feet in the summer up to about 8 feet during wintertime storage.

MR. WHITE: What were your initial coliforms just before disinfection?

DR. REYNOLDS: As I indicated, initial coliform concentrations ranged somewhere between a minimum of 10^3 up to about 10^4 or 10^5, an average of around 10^4.

MR. ZALEIKO, *Technical Associates:* Did you **get a chance to run any TOC-COD relationships on both the normal chlorine dosage as well as the extra heavy dosage?**

DR. REYNOLDS: Not that I recall. We ran a few TOC's, but not enough to establish a ratio between TOC and COD in the chlorinated effluent.

MR. ZALEIKO: So there were no parallel runs of TOC as well as COD?

DR. REYNOLDS: I do not recall any.

MR. ZALEIKO: Just spot checks on the TOC? I was wondering, since the Army gentleman in mentioning some of his problems, discussed TOC rather than COD relationships on his work rather than on yours. It can make a difference.

DR. HARVEY ROSEN: I may be a little embarrassed to ask this question, being a chemist, but have there been any studies done about the potential reactions with respect to toxicity when sulfur is present? It seems like SOCl and other things may be formed in lagoon effluents that would not be formed in most of the effluents that have been reported relative to the toxicity studies that tend to be aerobic when chlorinated.

DR. REYNOLDS: I am not aware of any. Now, some of the other gentlemen who are doing more on toxicity may know. By the way, the effluent was not anaerobic when it was chlorinated, it just contained a high degree of sulfide because of anaerobic conditions in the lagoon, but the total lagoon dissolved oxygen was still above 6.2 or 6.3 mg/l, even in the wintertime.

MR. CONVERY: I do know that the Health Effects Research Laboratory has tried to partition the organic concentrates from their RO concentrates of drinking water supplies, and I think they narrowed it down to 468 compounds. So, I am not sure that they can identify whether it is sulfonated organics or not.

7.

DECHLORINATION OF WASTEWATER: STATE-OF-THE-ART FIELD SURVEY AND PILOT PLANT STUDIES

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INTRODUCTION

Dechlorination is required in six of the County Sanitation Districts of Los Angeles County (CSDLAC) water reclamation plants (Figure 1). These plants are located upstream in the sewerage system to provide hydraulic relief for downstream sewers. Each day $2.5 \times 10^5 m^3$ (70 million gallons) of filtered chlorinated secondary effluent are discharged to the Rio Hondo and San Gabriel River flood control channels for reuse or overland flow to the ocean. Because most of the effluent is discharged to navigable waters, the upstream WRP's must, therefore, conform to National Pollutant Discharge Elimination System (NPDES) requirements.

Discharge requirements for CSDLAC facilities are set by the Los Angeles Region of the California Water Quality Control Board. Among other requirements, the total chlorine residual allowed in effluent discharged to navigable water is generally less than 0.1 mg/l. This requirement has been imposed primarily for the protection of fish life and other aquatic organisms in the receiving waters. Dechlorination with sulfur dioxide is employed in CSDLAC facilities to meet the chlorine standard.

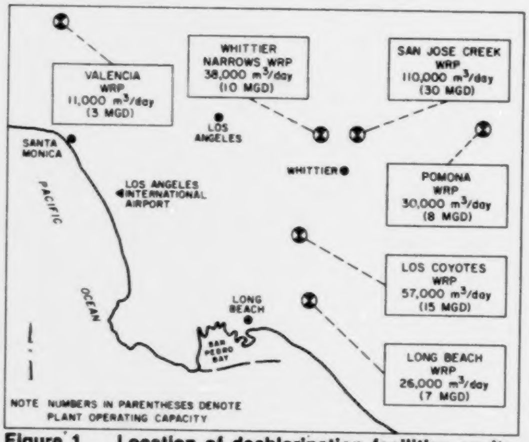

VALENCIA WRP 11,000 m³/day (3 MGD)
WHITTIER NARROWS WRP 38,000 m³/day (10 MGD)
SAN JOSE CREEK WRP 110,000 m³/day (30 MGD)
LOS ANGELES
SANTA MONICA
WHITTIER
POMONA WRP 30,000 m³/day (8 MGD)
LOS ANGELES INTERNATIONAL AIRPORT
PACIFIC
OCEAN
LOS COYOTES WRP 57,000 m³/day (15 MGD)
LONG BEACH
SAN PEDRO BAY
LONG BEACH WRP 26,000 m³/day (7 MGD)
NOTE NUMBERS IN PARENTHESES DENOTE PLANT OPERATING CAPACITY

Figure 1. Location of dechlorination facilities sanitation districts of Los Angeles County

As a result of the need for dechlorination in CSDLAC facilities, this study was, therefore, initiated with funds from the U.S. Environmental Protection Agency's Municipal Environmental Research Laboratory in Cincinnati, Ohio. The study consisted of a field survey of dechlorination facilities in the State and pilot plant studies.

Objectives of the field survey were to assess the effectiveness and reliability of full-scale dechlorination installations in California, which leads the nation in number of dechlorination facilities. Prime considerations of the survey were on reliability and methods of control for the sulfonation system and bacterial aftergrowth in the dechlorinated effluent. Information for the survey was gathered through questionnaires, which were mailed to superintendents of the dechlorination installations. Site visit follow-ups were made to some of the facilities.

In the pilot plant studies, the effluent was closely monitored for bacteriological, physical and chemical degradation effects after dechlorination. The three methods of dechlorination evaluated were sulfur dioxide (SO_2), granular activated carbon and holding pond. Emphasis in the study was on SO_2, because it is the most commonly employed method of dechlorination. Cost effectiveness on all three methods of dechlorination was examined.

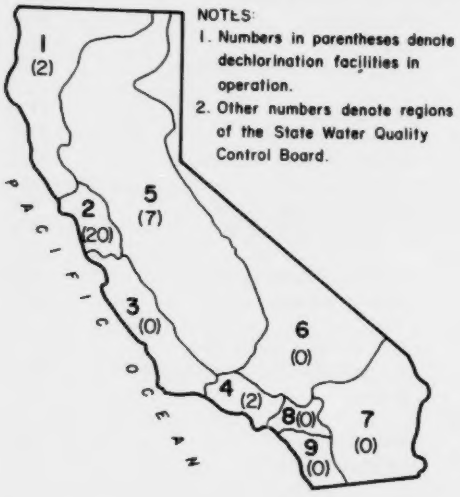
FIELD SURVEY

General Information

Disinfection of wastewater with chlorine can reliably meet the bacteriological standards for secondary treatment in California. However, it can result in a residual chlorine level that is toxic to fish. Available data indicate that a chlorine concentration of less than 0.1 mg/l is acutely toxic to fish. Dechlorination is therefore necessary for wastewater discharges that threaten the ecology of the receiving waters.

The dechlorination requirement for wastewaters in the State varies from one region to another. There are nine Regional Water Quality Control Boards in the State. Each Board sets and enforces the requirements for the region under its jurisdiction. For dechlorination, the chlorine discharge limits are set based on the existing qualities and beneficial uses of the receiving waters.

The nine California Regional Water Quality Control Boards (CRWQCB's) were contacted in May 1977 to determine the number of dechlorination installations in the State. From the list of treatment plants provided by CRWQCB, superintendents of individual plants were contacted by phone in June 1977, to inquire if dechlorination was practiced at their plants. Dechlorination was employed in 31 of the 60 facilities contacted. The remaining facilities were under design/construction. Questionnaires were then mailed to the 31 facilities employing dechlorination. Their total number in each of the regions is shown in parentheses of Figure 2.



NOTES:
1. Numbers in parentheses denote dechlorination facilities in operation.
2. Other numbers denote regions of the State Water Quality Control Board.

Figure 2. Map showing number of operating dechlorination facilities in California (1977)

Results from the field survey indicate that sulfur dioxide (SO_2) is the most widely used dechlorinating agent in California. It is more popular because of cost and ease of application. Equipment used for SO_2 is the same as that used for chlorine, which makes SO_2 application easily adaptable in most wastewater treatment facilities.

The chemical characteristics of the gas have also added to the attraction for the process. Reaction time of SO_2 and free chlorine or chloramines is very short. By-products of SO_2, such as sulfites and chlorides, have not been shown to be toxic to fish at normal levels encountered in dechlorination.

Dechlorination is employed mostly in the urban areas such as San Francisco (Region 2), Sacramento (Region 5), and Los Angeles (Region 4). The threat to fish and wildlife is most prevalent in these areas because of the voluminous amount of wastewater discharges. Receiving streams and rivers are incapable of assimilating the chlorine discharge from the treatment plants without endangering fish lives and other aquatic organisms.

Region 2 dechlorinates approximately $3.0 \times 10^{6} m^{3}$ (800 million gallons) of wastewater daily to protect fish and wildlife habitat in the San Francisco Bay estuaries. The Bay is one of the most important coastal estuaries in the State. Myriads of fish and wildlife species utilize the Bay habitats for feeding and nursery ground. The Bay also functions as the only drainage outlet for wastewaters in the region. Dechlorination is, therefore, necessary for wastewater discharges in the Bay.

Region 5 dechlorinates its wastewater to protect the fish and waterfowl habitat in the rivers. Region 4 dechlorinates the wastewater only in the inland plants to protect the fish in the low flowing rivers.

The North Coast (Region 1) has three facilities currently dechlorinating their effluent. Rapid growth has generated sufficient chlorinated wastewater to threaten the fish life in the area.

The Central Coast (Region 3) and San Diego (Region 9) utilize submarine ocean outfall to disperse off their wastewater discharges. No dechlorination is practiced in these areas.

The Colorado River Basin (Region 7) and Santa Ana (Region 8) are water scarce regions. Wastewater effluents are normally discharged to ephemeral streams which contained virtually no fish life. Dechlorination of wastewater is, therefore, not required in these regions.

The Lahontan Region (Region 6) disposes its wastewater via land disposal and, therefore, has no dechlorination facilities.

Questionnaire Responses

A summary of responses to the questionnaires is shown in Tables 1, 2, and 3. The questions are divided into three categories: general information, engineering design information, and operational information for the SO_2 dechlorination system. Responses to the questions are tabulated with respect to percent of the total dechlorination facilities surveyed.

TABLE 1. PROFILE OF DECHLORINATION FACILITIES IN SURVEY (1977)

Description	Percent of Total Responses*
(a) Startup date of dechlorination facilities	
- Before January, 1976	38.7
- After January, 1976	61.3
(b) Type of treatment preceding dechlorination	
- Primary	9.7
- Secondary	83.9
- Tertiary	6.4
(c) Average daily plant flow	
- Less than $2.3 \times 10^4 m^3/d$ (6 mgd)	68.0
- 6 to 10 mgd 2.3×10^4 to $3.8 \times 10^4 m^3/d$ (6-10 mgd)	16.0
- Greater than $3.8 \times 10^4 m^3/d$ (10 mgd)	16.0
(d) Sulfur dioxide capacity	
- 0 to 45.4 kg/d (0 to 100 lbs/day)	12.9
- 45.8 to 227 kg/d (101 to 500 lbs/day)	35.5
- Greater than 227 kg/d (500 lbs/day)	51.6
(e) Total coliform discharge standard	
- Less than or equal to 2.2/100 ml	22.6
- Less than or equal to 23/100 ml	16.1
- Less than or equal to 100/100 ml	9.7
- Less than or equal to 240/100 ml	41.9
- Others	9.7
(f) Total residual chlorine discharge standard	
- 0	58.1
- Less than or equal to 0.1 mg/l	29.0
- Greater than 2 mg/l	12.9

*Based on 31 respondents

Referring to Table 1, dechlorination facilities in the State are mostly installed after January 1976, thus reflecting the recent stringent requirement imposed on wastewater treatment facilities in California. They are mostly in secondary treatment plants with an average daily plant flow of 3.8 to 23 x $10^3 m^3/d$ (1 to 6 mgd) and SO_2 capacity greater than 230 kg/d (500 lbs/d). Secondary treatment is the minimum requirement for most wastewater discharges in the State. Consequently, most plants are in this category, which may account for the largest percentage of dechlorination found in secondary treatment.

Before dechlorination, the chlorine residuals in the treatment plants range from 2 to 10 mg/l. This is to meet the various disinfection goals shown in Table 1. The disinfection requirement of most wastewater treatment facilities is to meet either a 2.2 or less total coliform per 100 ml or 240 or less total coliform per 100 ml. The former limit is for facilities which discharge to waters for non-restricted recreation usage. The latter limit is for facilities which discharge to shellfish harvesting areas.

The total residual chlorine goal in most wastewater discharges is generally 0.1 mg/l or less. Several have reported total residual chlorine discharges ranging from 2 to 6 mg/l. In these plants, it was found that

TABLE 2. ENGINEERING DESIGN INFORMATION OF DECHLORINATION FACILITIES IN SURVEY (1977)

Description	Percent of Total Responses*
(a) Type of feed control system	
- Feedforward	87.1
- Feedback	9.7
- Feedforward and feedback	3.2
- Flow paced	27.4
- Residual control	27.4
- Flow and residual controls	45.2
- Pneumatic flow signal	6.5
- Electronic flow signal	93.5
- Pneumatic dosage signal	9.7
- Electric dosage signal	90.3
- Gap residual controller	16.1
- Proportional and reset controller	25.8
- None	58.1
- With multiplier	35.5
- Without multiplier	64.5
- With adjustable slope factor	9.6
- Without adjustable slope factor	90.4
(b) Contacting method	
- SO_2 injected in mixing chamber	32.3
- SO_2 injected in outfall pipe	67.7
- Reaeration provided after dechlorination	3.2
- Reaeration not necessary after dechlorination	96.8
- pH adjustment provided after dechlorination	3.2
- Others	12.9

*Based on 31 respondents

the discharge limitation for chlorine is not required until several miles downstream. Consequently, the plant can discharge a higher level of chlorine. Due to normal chlorine dissipation along the channel, virtually all the chlorine is removed by the time it reaches the required sampling point.

Table 2 summarizes the engineering design information gathered in the survey. On the method of SO_2 feed, most responses showed a feed forward type system with primary control based on flow and secondary control based on residual levels at the chlorine contact chamber effluent. The secondary signal must be feed forward rather than a closed loop, since currently available residual chlorine analyzers are not capable of monitoring and controlling on the basis of a dechlorinated effluent. Both the flow and dose signals are electronic rather than pneumatic, thus reflecting the newness of the equipment. Controllers, multipliers, and adjustable slope factors are not used in most installations. These devices have been used to fine-tune the SO_2 requirement for dechlorination.

Responses on the SO_2 contacting methods showed injection of the gas largely occurs through diffusers with no mixing chamber provided. Reaction of SO_2 and chlorine have been shown to be rapid from rate equation analyses. Neither reaeration nor pH adjustment has been found necessary in most installations, which is contrary to what has been anticipated by many people due to the reducing reactions of SO_2.

TABLE 3. OPERATIONAL INFORMATION OF DECHLORINATION FACILITIES IN SURVEY (1977)

Description	Percent of Total Responses*
(a) Is dechlorination system operated 24-hrs daily?	
- Yes	93.5
- No	6.5
(b) What is the desirable SO_2:Cl_2 ratio employed?	
- 1 or less	74.2
- greater than 1	25.8
(c) Is overdosing necessary to meet standard?	
- Yes	87.1
- No	12.9
(d) Is SO_2 feed control system reliable?	
- Yes	58.1
- No	41.9
(e) Will system handle drastic fluctuation of residual chlorine?	
- Yes	50.0
- No	50.0
(f) Is biological aftergrowth observed after dechlorination?	
- Yes	6.5
- No	93.5

*Based on 31 respondents

Table 3 shows a summary on the operational information of the plant survey. Approximately 94 percent of the plants reported operating the dechlorination system continuously for 24 hours daily. The remaining 6 percent operate the system intermittently, turning on the SO_2 only at high chlorine flows. Approximately 74 percent attempt to operate the SO_2 close to the theoretical SO_2: chlorine residual ratio of 0.9; however, most (87 percent) find overdosing necessary to achieve the low or zero chlorine residual standard. Approximately 97 percent reported reaeration to be unnecessary after SO_2 dechlorination, and 97 percent found no pH adjustment required of the dechlorinated effluent (Table 2). Biological aftergrowth was not observed in 94 percent of the installations. Approximately 58 percent felt the SO_2 feed control system used in the plant was reliable.

Site Visits

Table 4 lists the dechlorination facilities visited on the trip. The facilities are located in the San Francisco, Sacramento, San Jose, Santa Rosa, and Los Angeles areas. The purpose of the trip was to study various SO_2 feed control methods and interview operators regarding operation of their system. A total of 17 installations were visited. All the wastewater treatment plants visited have secondary treatment. The plant flow capacities ranged from 1.1×10^4 to $3.0 \times 10^5 m^3/d$ (3 to 80 mgd). The SO_2 capacities ranged from 200 to 6900 kg/d (450 to 15,200 lbs/d).

Figure 3 shows the most common method of SO_2 dechlorination observed in the site visits. In the system, a feed forward residual signal and a feed forward flow signal are fed to the sulfonator. These two signals are sometimes combined into a product signal through an electronic multiplier before feeding to the sulfonator. This is done to avoid having to excessively overdose the chlorinated effluent with SO_2.

To improve the system further, an electronic ratio controller is adopted after the multiplier in some facilities. The ratio station maintains the desired ratio between the flow and residual signals combined in the multiplier. This provides a more precise control on the SO_2 dosage requirement for dechlorination.

Alternate methods have been devised to overcome inability of the chlorine analyzer in providing a feed-

TABLE 4. DECHLORINATION FACILITIES VISITED IN SURVEY (1978)

Dechlorination Facilities	Operated by	SO_2 Capacity, kg/day (lbs/day)	Daily Avg. Flow, m^3/d (MGD)
City Main WTP	City of Sacramento	6,900 (15,200)	1.8×10^5 (48)
San Jose/Santa Clara WPCP	City of San Jose	3,450 (7,600)	3.0×10^5 (80)
Northeast WRP	County of Sacramento	2,724 (6,000)	5.7×10^4 (15)
Arden WTP	County of Sacramento	1,725 (3,800)	1.9×10^4 (5)
San Pablo San. WPCP	San Pablo San. Dist.	908 (2,000)	3.0×10^4 (8)
Main Treatment Plant	San Rafael San. Dist.	908 (2,000)	1.1×10^4 (3)
Hayward WTP	City of Hayward	908 (2,000)	4.5×10^4 (12)
Pomona WRP	L.A. Co. San. Districts	908 (2,000)	2.3×10^4 (6)
Harold May WQCP	City of Palo Alto	863 (1,900)	1.1×10^5 (30)
Laguna WTP	L.A. Co. San. Districts	454 (1,000)	1.1×10^4 (3)
Valencia WRP	L.A. Co. San. Districts	454 (1,000)	1.1×10^4 (3)
Cordova WTP	County of Sacramento	431 (950)	7.6×10^3 (2)
Richmond WPCP	City of Richmond	409 (900)	2.7×10^4 (7)
Irvington WTP	Union Sanitary Dist.	227 (500)	2.3×10^4 (6)
Newark WTP	Union Sanitary Dist.	227 (500)	1.9×10^4 (5)
Alvarado WTP	Union Sanitary Dist.	227 (500)	1.1×10^4 (3)
West College WTP	City of Santa Rosa	204 (450)	1.5×10^4 (4)

back signal to the sulfonator. Figure 4 shows a schematic of such a system.

In alternate No. 1, a two-stage method of dechlorination is used. Analyzer No. 1 is used to instruct Sulfonator No. 1 to dechlorinate to a 10:1 ratio of the discharge limit. The analyzer performs best within a 10 to 1 setting. Calibration is maintained because of the continuous presence of chlorine residual in the effluent. Sulfonator No. 2 is then used to remove the remaining residual chlorine. Because the residual chlorine has been reduced to 1 mg/l or less in the first stage, excessive overdose of the SO_2 with sulfonator No. 2 is avoided.

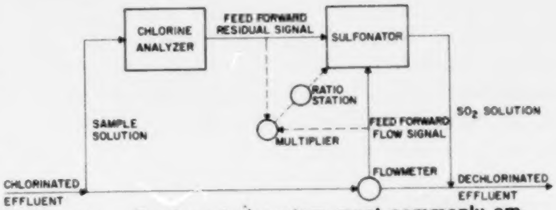


Figure 3. Feed control system most commonly employed in sulfur dioxide dechlorination facilities in California.

In alternate No. 2, a biased residual chlorine signal is sent through the analyzer to keep it in calibration. A feedback residual signal from the dechlorinated effluent greater than the biased signal signifies incomplete dechlorination. The SO_2 is paced to dose proportional to any signal greater than the biased signal.

The simple feed forward SO_2 feed control system is adequate for most dechlorination installations. It requires a small capital investment and offers simplicity of controls. Disadvantage of the system is that SO_2 overdosing is necessary to accomplish disinfection. Overdosing (SO_2) cost is a significant factor in large dechlorination installations. Alternate SO_2 feed control systems are beneficial for the large installation. It reduces the SO_2 overdose requirement and hence the operating chemical cost.

The weakest link in the SO_2 feed control system is the chlorine residual analyzer. The measuring electrode of the analyzer loses its sensitivity rapidly in a dechlorinated effluent. The presence of chlorine residual helps prevent oxidation on the electrode. Abrasive grits in the measuring cell block are incapable of preventing oxides from forming on the electrode in the absence of chlorine.

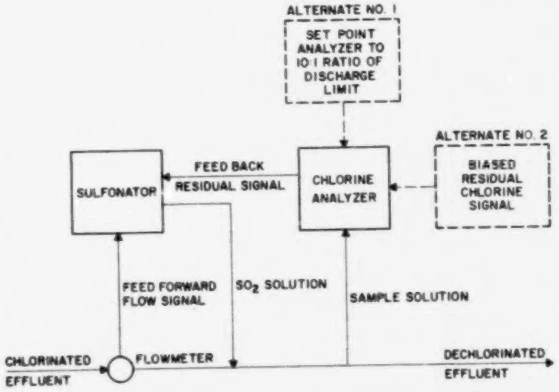


Figure 4. Feed control systems used in dechlorination facilities to avoid excessive SO_2 overdose.

Figure 5 shows a schematic of a residual chlorine sampling cell. Measuring of the chlorine takes place in the measuring cell block. The platinum is the reference electrode and the copper is the measuring electrode. A small D.C. current is produced in the presence of free chlorine or iodine proportional to its concentration. This current is measured by a recording ammeter in terms of mg/l of chlorine. The efficient operation of the analyzer is dependent upon several factors, all of which are equally important. The cell block and electrodes have to be free from biological or chemical fouling of the electrodes. Abrasive grit and installation of filters have been successful to some extent in preventing contamination of the electrodes. The buffer solution must be maintained since the analyzer is calibrated to read accurately at a specific pH. The flow and pressure of the sample water must be fairly constant, since they affect cell current production. A malfunction in any of the above conditions will give a false reading and result in underdosing or overdosing of the SO_2.

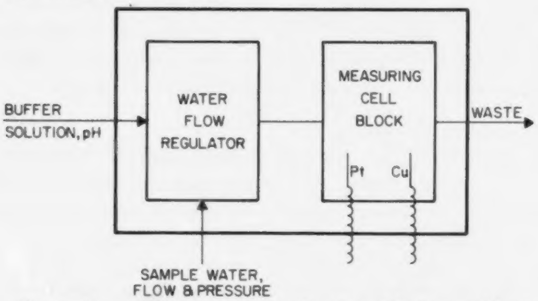


Figure 5. Critical components of a chlorine residual analyzer

Poor analyzer performance has been found in SO_2 dechlorination installation where there are high industrial waste discharges. Plugging of the measuring cell block frequently occurs. Installation of filters preceding the cell block did not improve the analyzer reliability.

Operators' comments on the reliability of the sulfonation system have been generally satisfactory. Most felt a slight overdose of the effluent with SO_2 is necessary to meet the discharge requirement of 0.1 mg/l or less chlorine residual consistently. Biological growth in the dechlorinated effluent was not observable in most plants since the effluents were usually discharged in a submerged outfall pipe or intermingled with the receiving waters containing natural fresh water slime growth. However, laboratory reports from these plants indicated an increase in total coliform after dechlorination. As a result, coliform discharge limitations were usually exceeded at the point of discharge. The Regional Water Quality Control Board has now allowed the treatment plants to sample before dechlorination to meet the disinfection standard.

A continuous monitoring of the dechlorinated effluent is required in some cases for compliance with the discharge standard of CRWQCB. However, since a continuous recording of zero chlorine on the chart is not possible because of the limitation of the analyzer, the CRWQCB has allowed monitoring of the dechlorinated effluent for residual compliance once every hour. The analyzer monitoring the chlorinated effluent is used to monitor the dechlorinated effluent for residual chlorine compliance. Because this is done only a few minutes every hour, the analyzer is kept in calibration.

PILOT PLANT STUDIES

General Information

Figure 6 shows a schematic of the three dechlorination systems evaluated at the Pomona Research Facility. Unchlorinated secondary effluent from the Pomona Water Reclamation Plant was pumped to the Pomona Research Facility. The effluent was then chlorinated in a pilot system for two hours and then fed to the dechlorination systems. The sulfur dioxide and hold-

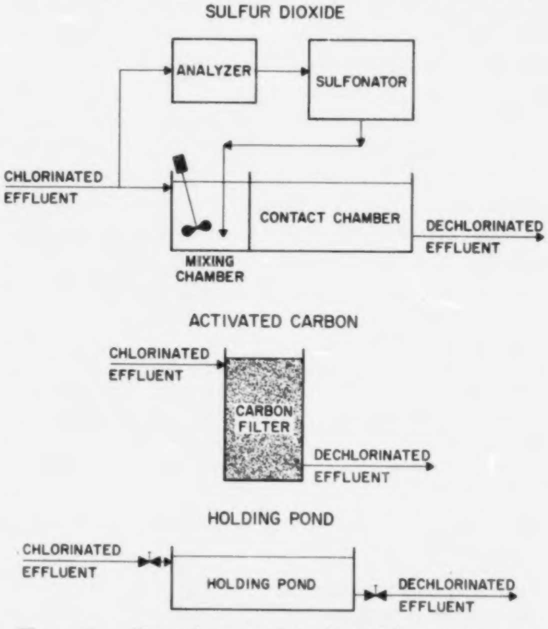


Figure 6. Flow diagram of dechlorination pilot plant systems at Pomona, California.

ing pond studies were conducted recently. The activated carbon study was done several years ago as part of another study.

DESCRIPTION OF PILOT PLANT

Sulfur Dioxide

The sulfur dioxide (SO_2) feed control system used in the pilot studies was obtained from a previous chlorination study. It employed only a feed forward chlorine residual signal to dechlorinate the effluent. The SO_2 was paced in proportion to the chlorine measured in the effluent. Because the flow was held constant, no flow signal was required. A multiplier unit and ratio station was not considered for the pilot systems because the objective here was to study the effects on water quality after dechlorination. Similarly, the other alternate schemes found in the field survey were not considered.

The chlorine residual of the effluent feeding through the SO_2 dechlorinations system averaged 7 mg/l. This was continuously monitored by a chlorine analyzer and a recorder. An electronic signal was sent from the analyzer to the sulfonator to control SO_2 dosage based on chlorine residual. The sulfonator was readjusted to feed SO_2 in direct proportion to the chlorine in the effluent.

Chlorinated effluent flow through the SO_2 dechlorination system averaged 95 l/min (25 gpm). A steel tank measuring 13.7m by 0.9m (45 feet by 3 feet) deep was used as a mixing and contact chamber. SO_2 was added to the chlorinated effluent in the mixing chamber at ratios ranging from 1-2 mg/l of SO_2 to 1 mg/l of chlorine residual. The ratio employed in each experiment was dependent on the SO_2 required to remove the chlorine in the pilot studies. Theoretical contact time in the mixing chamber was 10 minutes. From the mixing chamber, the dechlorinated effluent was fed through the contact chamber to study bacterial aftergrowth and physical-chemical degradation of the effluent after dechlorination. The theoretical contact time in the chamber was two hours.

TABLE 5. AVERAGE* WATER QUALITY CHARACTERISTICS OF CHLORINATED ACTIVATED SLUDGE EFFLUENT IN PILOT PLANT STUDIES

Parameters	Before SO_2 $DeCl_2$	After SO_2 $DeCl_2$
pH	7.2	7.2
Dissolved Oxygen (mg/l)	4.8	6.1
Sulfite (mg/l SO_3)	0.0	2.6
Alkalinity (mg/l)	172.0	164.0
Total Dissolved Solids (mg/l)	499.0	471.0
Total Residual Chlorine (mg/l)	5.0	0.0

*Average of 56 samples

Activated Carbon

Key operating parameters of the activated carbon dechlorination system may be summarized as follows. The activated carbon was a Calgon Filtrasorb 400 with 12 X 40 U.S. standard sieve size. It was contained in a steel tank measuring 1.8m (6 feet) in diameter and 4.9m (16 feet) high. Carbon depth in the column was (3.1 m) 10 feet. Chlorinated secondary effluent flowed downwards through the carbon at a loading rate of $0.28 m^3/min/m^2$ (7 gpm/ft^2). The empty bed detention time at this rate was 10 minutes. Sampling parts were provided at various depths in the column.

Holding Tank

An open steel tank of the same size as the SO_2 dechlorination tank was used. Chlorinated secondary effluent at 5 mg/l chlorine residual was fed to the holding tank and held until the chlorine had dissipated.

Samples in all three systems were collected before and after dechlorination. In addition, for the sulfur dioxide and holding pond experiments, samples were also collected at several periods after dechlorination. Both 24-hour composite and grab samples were collected during the pilot plant operation. The composite samples were analyzed for water quality. The grab samples were analyzed for bacterial aftergrowth.

Pilot Plant Results

The pilot plant studies addressed the question of water quality and bacteria regrowth after dechlorination. All three systems were evaluated with emphasis on the sulfur dioxide dechlorination system.

TABLE 6. AVERAGE* WATER QUALITY CHARACTERISTICS OF CHLORINATED ACTIVATED SLUDGE EFFLUENT IN PILOT PLANT STUDIES

Parameters	Before Carbon $DeCl_2$	After Carbon $DeCl_2$
Total COD (mg/l)	19.0	13.0
Dissolved COD (mg/l)	16.0	10.0
Color (units)	14.0	4.0
Turbidity (Jtu)	1.8	2.4
Suspended Solids (mg/l)	2.4	2.1
Total Residual Chlorine (mg/l)	8.0	0.0

*Average of 52 samples

Water Quality Effects

Tables 5 and 6 summarize the water quality characteristics before and after dechlorination for the sulfur dioxide and activated carbon dechlorination systems, respectively. No data are shown for the holding pond experiments since no change other than the chlorine content was observed in the effluent.

Referring to Table 5, no adverse effects on the water quality were found after dechlorination with SO_2. The pH remained relatively unchanged at 7.2. This may have been due to the high alkalinity (172 mg/l) in the Pomono effluent. Dissolved oxygen was increased from 4.8 mg/l to 6.1 mg/l. The increase could be attributed to turbulent mixing in the dechlorination chamber. The average sulfite level in the effluent was 2.6 mg/l due to a slight overdose of SO_2. Alkalinity was reduced from 172 mg/l to 164 mg/l possibly due to reaction with the SO_2. This decrease may have resulted in the reduction of total dissolved solids from 499 mg/l to 471 mg/l.

Dissolved oxygen depletion and pH reduction after SO_2 addition have been anticipated by many people because of the reducing quality of SO_2. These were not observed either in the pilot plant results or in the data collected in the plant full scale survey. A brief study during the pilot plant runs shows an SO_2 overdose as high as 50 mg/l is necessary to incur any significant change in the above parameters.

For the activated carbon system (Table 6) no adverse effects on the water quality were observed in the dechlorinated effluent. The effluent quality after chlorine by the carbon did not seem to alter the ability of the carbon for removing COD (total and soluble) and color.

TABLE 7. JAR TEST RESULTS IN BACTERIA AFTERGROWTH AFTER DECHLORINATION

Experiment No.	Chlorinated Effluent (2 hrs)	Dechlorinated in Sample Bottle (10 min)	Dechlorinated in Pilot Plant Tank (10 min)
1	2	2	49
2	<2	<2	79
3	5	2	33
4	<2	2	49
5	2	2	33
6	2	<2	49
7	2	<2	490

Note: All results expressed at total coliform MPN per 100 ml

Bacteriological Aftergrowth

Bacteria measured for this phase of work included total and fecal coliforms, total aerobic plate count at 35°C, fecal streptococci, and *Salmonella*. All tests were performed in accordance with *Standard Methods* (1) except for the *Salmonella spp.* which were isolated using methods recommended by the District's microbiology laboratory and the EPA project officer.

Figure 7 shows typical total coliform results for the dechlorination systems. An increase in total coliforms shortly after dechlorination was consistently observed in both the filtered and unfiltered dechlorinated secondary effluents. The origin of the increase was not known. It was presumed that the increase might be due either to a reactivation of the bacteria injured during chlorination or to contamination from the tank. Contamination from the air was not considered likely since the sulfonation chamber was covered throughout the tests.

Jar tests were performed in the laboratory to determine if the total coliform increase could be attributed to repair of injured bacterial cells. A chlorinated secondary effluent sample from the pilot plant was collected in a sterilized beaker and then dechlorinated with SO_2 solution (sulfurous acid). Table 7 is a summary of the results. No increase in total coliform density was observed in the beaker after dechlorination.

The dechlorination systems were subsequently investigated for possible sources of contamination. For the SO_2 system, the mixing and contact chambers were thoroughly cleaned and disinfected. For the carbon system, freshly regenerated carbon was used. The holding tank was also thoroughly cleaned and disinfected prior to the experiment. The systems were then put in operation and the dechlorinated effluent samples collected at 1, 2, 3, and 4 days after startup. For the holding tank, the samples were collected as soon as the chlorine had dissipated.

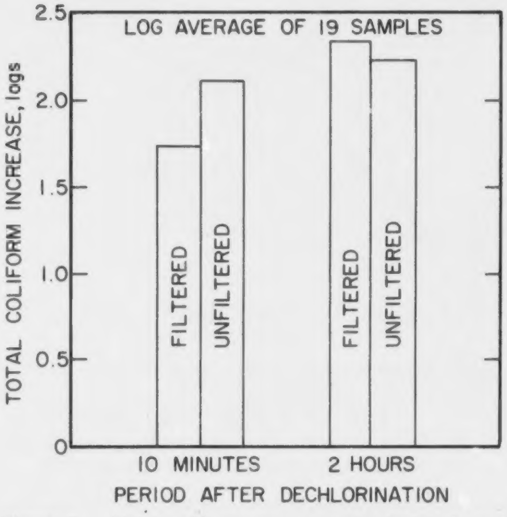


Figure 7. Total coliform responses in dechlorinated secondary effluents of pilot plant studies

Figure 8 shows a summary of typical results for all three systems. Total coliforms were increased by about two log units in all three dechlorination systems within the first four days. It is not known why the increase leveled off after four days. The gradual total coliform increases suggest that the source of contamination may be due to a buildup of bacterial growth in the dechlorination systems. Slime and scum growth were consistently observed in the sulfonation chamber a few days after operation. It is presumed that the same buildup may have occurred in the activated carbon column, although this was not observable because of the enclosed steel tank used. Growth in the holding pond was also observed after the chlorine residual had dissipated.

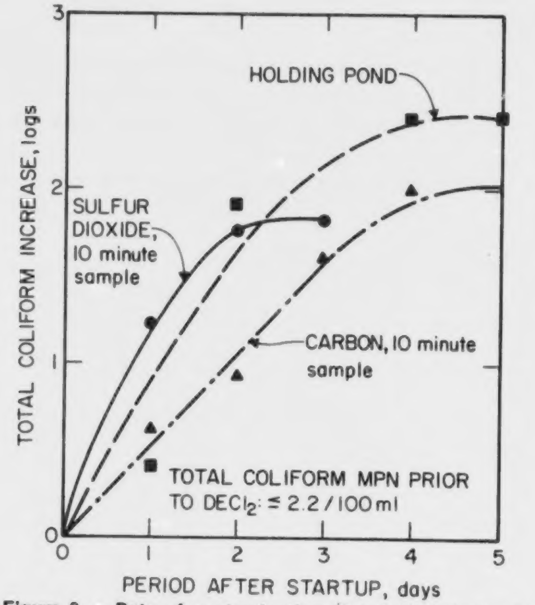


Figure 8. **Rate of contamination after initial startup in clean dechlorination pilot plant systems.**

Besides the total coliform analyses, other bacteriological analyses were performed. These tests were conducted only in the SO_2 experimental phase. Figure 9 is a summary of the results. Increases were found in the fecal coliform and total plate count populations. These increases (less than 0.5 log unit) were significantly lower than those found for the total coliform populations. No change in the fecal streptococci count was found after dechlorination.

To pursue the increase of the total and fecal coliforms further, photomicrographs of slime, scum, and foam samples collected at the head end of the dechlorination chamber were taken. The slime is the slippery substance that attaches to the insides of the tank; the scum is the deposit at the bottom of the tank; and the foam is the flocculant material that floats on the surface of the tank. The objective of the test was to ascertain the bacteriological make-up of these substances to gain insight into the possible source of contamination.

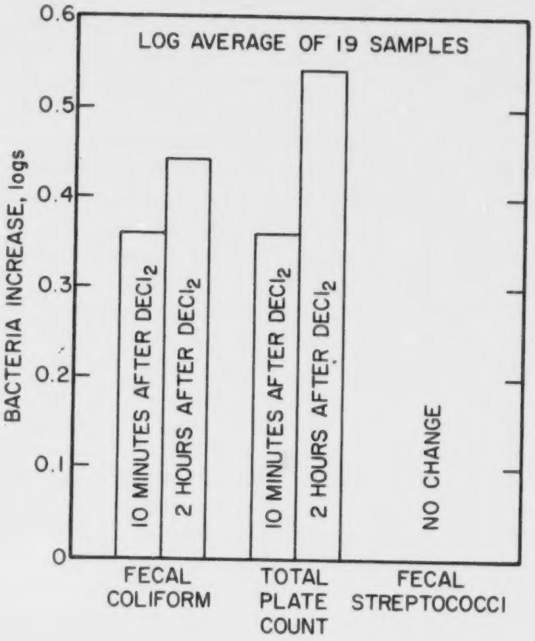


Figure 9. **Bacteria responses in dechlorinated effluents of pilot plant studies**

Photomicrographs of the slime, scum, and foam samples showed matrices of algae growth at different stages of the life cycle. These growths are sometimes referred to as filamentous slime. They are ideal sites for coliform attachment and growth, given the nutrients and moderate temperature of the wastewater. Except for the sulfur bacteria which were identified as *Beggiatoa,* no specific microorganisms were identified in the samples.

Table 8 summarizes the total and fecal coliform results for the slime, scum, and foam materials in the dechlorination chamber. The highest total and fecal coliform counts were found in the scum with MPN's of 1.9×10^8/gm and 2.8×10^6/gm material, respectively.

TABLE 8. ANALYSES OF AFTERGROWTH MATERIALS IN DECHLORINATION CHAMBER OF PILOT PLANT

Organisms	Slime (MPN/100 ml)	Scum (MPN/g-mat'l)	Foam (MPN/g-mat'l)
Total Coliform	39	1.9×10^8	6.0×10^6
Fecal Coliform	23	2.8×10^6	6.0×10^5

The foam had MPN's of 6.0 x 10^6/gm material and 6.0 x 10^5/gm material for the total and fecal coliform, respectively. The slime had MPN's of 39/100 ml and 23/100 ml for the total and fecal coliform, respectively.

In addition to the above bacteriological tests, *Salmonella* analyses were also performed. Two enrichment media were employed to isolate the *Salmonella;* namely, dulcitol selenite broth (DSE) and tetrathionate broth (TB). Both have been shown to be effective media for enriching *Salmonella*. Following enrichment, isolation and confirmation of *Salmonella* colonies were carried out. Isolation media used were Xylose Lysine Desoxycholate (XLD) and Bismuth Sulfite (BS). Confirmation of typical colonies was performed biochemically using Lysine Iron Agar, Triple Sugar Iron Agar, and Urea Broth, and serologically using *Salmonella* polyvalent-O antiserum. A total of 26 tests were conducted. All were qualitative tests since low numbers of *Salmonella* were anticipated due to chlorine disinfection in the effluent. The results provided an estimate of sampling volume size for the quantitative tests performed later.

TABLE 9. *SALMONELLA* ISOLATIONS IN EFFLUENTS OF PILOT PLANT STUDIES WITH KNOWN SALMONELLA INPUT

Sample Date	Chlorinated Activated Sludge Effluent	Dechlorinated Activated Sludge Effluent
3/29/78	+	-
4/05/78	+	-
5/08/78	+	-
5/31/78	+	-
6/19/78	+	-
6/28/78	+	-
7/06/78	+	+
7/10/78	+	-
7/12/78	+	+
7/17/78	+	-
7/24/78	+	+
7/28/78	+	-
8/02/78	+	-
8/23/78	+	-
8/25/78	+	-

Table 9 summarizes the qualitative results of the *Salmonella* tests. Only results where *Salmonella* were isolated in the unchlorinated effluent are presented. Of the 26 unchlorinated samples, *Salmonella* were isolated in 15. Of these samples, only three, or 20 percent of the samples, showed *Salmonella* presence after dechlorination. It is not known if the *Salmonella* presence was due to regrowth after dechlorination.

Table 10 lists the quantitative results and the sample volume filtered for each corresponding test. The procedure used was the same as the qualitative method except that 16-24 liter samples were filtered; the filters were placed in 10 ml enrichment broth and homogenized in a Waring blender; and the resulting homogenate tested by the MPN procedure, using tetrathionate broth at 37°C. No *Salmonella* were found in the dechlorinated effluent even though positives were found in the unchlorinated effluent. The *Salmonella* counts for the unchlorinated secondary effluent ranged from 8 to 40 per liter sample. The *Salmonella* counts after dechlorination were less than 0.06 per liter sample. The lowest detection limit for these tests was 0.05 per liter sample.

TABLE 10. QUANTITATIVE ANALYSES FOR *SALMONELLA* IN EFFLUENTS OF PILOT PLANT STUDIES

Sample Date	Chlorinated Activated Sludge Effluent (*Salmonella*/liter)	Dechlorinated Activated Sludge Effluent (*Salmonella*/liter)
8/15/78	10	<0.05
8/16/78	8	<0.05
8/17/78	40	<0.06

The results showed that prevention of slime growth in the dechlorination test chambers is virtually impossible. The growth appears to be similar to that found in natural streams. These slime growth in dechlorinated effluent may be attributed to available nutrients in wastewater, the moderate ambient temperature and the lack of a toxic chlorine residual. It is felt that the presence of chlorine is an important factor in deterring this growth. This was learned from the site visits to dechlorination installations in Northern California.

COST ESTIMATES

Cost estimates for three dechlorination systems were derived using the information gathered in the study. These were the SO_2, activated carbon, and holding pond dechlorination systems. In deriving the costs, it was assumed that the wastewater treatment plant capacity is 38000m^3/d (10 mgd) with 5 mg/l residual chlorine to be dechlorinated. The residual chlorine concentration is based on a requirement to meet the State's disinfection standard of 2.2 or less total coliform MPN per 100 ml.

Table 11 summarizes results of the cost analyses. The amortization period for the capital costs was 15 years at an annual interest rate of 8 percent.

Sulfur Dioxide

Total capital cost for the system was $100,000. The cost includes two 390 kg/d (850 lbs/d) sulfonators with one used as standby, control building, piping and instrumentation. The sulfonator was sized based on a SO_2:Cl_2 ratio requirement of 2:1. In the pilot studies at Pomona this ratio was found necessary when high alkalinity of 200 mg/l was found in the chlorinated secondary effluent. The theoretical SO_2:Cl_2 ratio requirement is 0.9:1.0. No provision was made for a contact chamber since the SO_2/Cl_2 reactions are rapid as found in the pilot plant studies and treatment plant survey.

TABLE 11. COST ESTIMATES FOR A 38000m³/d (10 mgd) ACTIVATED SLUDGE DECHLORINATION FACILITY

Dechlorination Process	Capital Cost ¢/3.8m³ (¢/1000 gal)	Operating Cost ¢/3.8m³ (¢/1000 gal)	Total Cost ¢/3.8m³ (¢/1000 gal)
Sulfur Dioxide	0.4	1.3	1.7
Activated Carbon	2.3	10.5	12.8
Holding Pond	5.4	0.4	5.8

Total operating cost for the SO_2 dechlorination system was $50,000 per year. The cost was based on a chemical cost of $220 per ton of SO_2 and an annual salary of $15,000 for one operator.

Total cost for the SO_2 dechlorination system was 1.7 cents to treat 3.8m³ (1000 gallons) of chlorinated secondary effluent. Of this total, 76.5 percent was for operation and maintenance and 23.5 percent was for capital recovery.

Carbon Dechlorination

Total capital cost for the system was $400,000. Capital cost for the carbon includes the dechlorination column and holding tanks, regeneration furnace, instrumentation, piping, valves, pumps and initial carbon load. The carbon filters were sized based on a chlorine adsorption capacity of 0.068 kg (0.15 lbs) Cl_2 per 0.45 kg (1 lb) carbon for 1.8m (6 feet) of carbon depth. Loading rate for the carbon was 0.28m³/min/m² (7.1 gpm/ft²) for an empty bed detention time of 10 minutes. Calgon Filtrasorb 400 carbon with U.S. standard mesh size of 12 x 40 was used. Cost of this carbon was 58 cents per 0.45 kg (1 lb) carbon.

Total operating cost for the carbon dechlorination was $400,000 per year. Operating cost for the carbon was based on 45 days regeneration frequency. Regenerated carbon cost was 35 cents per 0.45 kg (1 lb) C., based on information generated at the Pomona Carbon Filtration Plant. Carbon loss during regeneration was assumed at 6 percent. One full time operator was considered for the system. Operators for the regeneration system were not considered since the regenerated carbon cost includes the operators' cost.

Total cost for the carbon dechlorination system was 12.8 cents to treat 3.8m³ (1000 gallons) of chlorinated secondary effluent. Of this total, 80 percent was for operation and maintenance and 20 percent was for capital recovery.

Holding Pond

Total capital cost for the holding ponds was $1,700,000. This includes excavation and lining the pond with 10 cm (4 inch) gunite. The pond was sized to allow a detention time of five days for the 5 mg/l chlorine residual to dissipate. This dissipation time was derived in the pilot studies and is based on abundant sunlight during most part of the day as in Southern California.

Total operating cost for the holding pond was $15,000 per year. This was to pay the salary for one operator.

Total cost for the holding pond dechlorination system was 5.8 cents per 3.8m³ (1000 gallons) of dechlorinated effluent. Of this total, 93 percent was for capital and 7 percent was for operation and maintenance.

The cost data confirm the economic justification of the municipalities in California for using SO_2 over the other dechlorination processes. The cost presented is for a 3.8 x 10⁴m³/d (10 mgd) plant. In an EPA cost analysis (2) for SO_2 dechlorination, the cost for a 3.8 x 10³m³/d (1 mgd) dechlorination facility was shown to be many times higher than for a 3.8 x 10⁴m³/d (10 mgd) plant. The same report showed the cost for a 3.8 x 10⁵m³/d (100 mgd) plant to be less than for the 3.8 x 10⁴m³/d (10 mgd) plant.

Dechlorination by the other methods such as carbon or holding pond does not seem to be economically feasible. Water quality effects and bacteriological aftergrowth after dechlorination were observed to be similar to SO_2 dechlorination.

The use of carbon for dechlorination may be attractive when removal of trace organics is required. In the pilot carbon studies, it was found that the adsorption capacity of carbon for the organic material is only slightly reduced by the presence of chlorine. The total dissolved COD removal was reduced by only 5 percent in the carbon treating chlorinated effluent after 90 days onstream.

The use of a holding pond for dechlorination may be attractive in a region where land is plentiful and inexpensive. The presence of sunlight is an important criterion for this process installation. An advantage of the system is that virtually no mechanical equipment needs to be used. Considerable savings in operating cost and many years of services are obtainable with this system.

CONCLUSIONS

1. Sulfur dioxide (SO_2) is the most preferred method for dechlorination of wastewater in California.
2. The feedforward method of control with flow as the primary signal and residual chlorine as the secondary signal is most commonly used in SO_2 dechlorination facilities.
3. Overdosing the chlorinated wastewaters with SO_2 is essential to accomplish consistent dechlorination.
4. Excessive overdose of SO_2 can be avoided by using discrete instruments and alternate methods of feed.
5. Except for the residual chlorine analyzer, the equipment in an SO_2 feed control system is reliable.
6. The analyzer is the weakest link in a SO_2 feed system. Most analyzers manufactured today are incapable of maintaining calibration in the absence of chlorine.
7. No significant physical-chemical degradation of the effluent was found after dechlorination with SO_2. Depletion of dissolved oxygen or change in pH was not observed in the pilot studies at SO_2 dosage to residual chlorine ratio of 2:1.
8. Bacteriological aftergrowth in some microorganism populations was found after dechlorination. This was observed predominantly for the total coliform group. Some increases in fecal coliforms and other bacteria (as detected in the total plate count) were also found to increase in the dechlorinated secondary effluent. *Salmonella* was not detected in most of the samples. Fecal streptococci in the effluent remained relatively unchanged after dechlorination.
9. The bacterial increase in the dechlorinated effluent is attributed to contamination from bacteriological aftergrowth in the dechlorination chamber rather than reactivation of injured bacterial cells.
10. Sulfur dioxide dechlorination is a more cost-effective process than either activated carbon or holding pond.
11. Similar effects on water quality and bacteriological contamination of the dechlorinated effluent were found for the activated carbon and the holding pond as with the SO_2.

ACKNOWLEDGEMENTS

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The cooperation from respondents of the questionnaires in the survey and plant personnel in the site visits is greatly appreciated.

George C. White gave invaluable assistance in the field survey.

Credits for this research are extended to staff of the Pomona Research Laboratory and the San Jose Creek Microbiology Laboratory.

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DISCUSSION

MR. RON SOLTIS, *Washington Suburban Sanitary Commission:* How long have the carbon columns been in service?

MR. GAN: You mean the carbon that we used for the dechlorination study?

MR. SOLTIS: Yes, for the dechlorination studies.

MR. GAN: Those are freshly regenerated carbon.

MR. SOLTIS: Do you have any facilities in California that use this as a routine method of dechlorination?

MR. GAN: Not that I know of.

MR. VENOSA: You established a cost of dechlorination. What did you assume to be the chlorine residual to establish your costs?

MR. GAN: The chlorine residual is 5 mg/l. This is the condition for the Pomona Water Reclamation Plant.

MR. BUONOMO, *Envirotech, National Sonics, Delta Scientific:* I want to comment on your statement that at close to zero residual chlorine you had difficult problems monitoring and therefore maybe that explains why closed loop control was only 58% effective. Your statement does hold for naked electrode amperometric type technique for monitoring residual chlorine. We have been using that for the last 10-15 years. With the polarographic membrane technique, you can go to zero or close to zero residuals and have excellent recovery capabilities. That is more than a statement of theory. On the West Coast, six utilities, funded at Trojan Nuclear, Portland General Electric, proved that they could achieve closed loop control and approach zero residual using the polarographic membrane. Incidentally, the maximum discharge limitations there were 0.05 parts per million.

MR. ED JONES, *District of Columbia Government, Blue Plains:* Do you think that 3 parts per billion is reasonable treatment for chlorine residual?

MR. GAN: You want my opinion on it?

MR. JONES: Yes, that has been the initial decision Region III imposed on Blue Plains.

MR. GAN: No, I do not think it is a reasonable level.

8.

ROUNDTABLE DISCUSSION OF CHLORINATION/DECHLORINATION

Participants in Chlorination/Dechlorination Roundtable

John J. Convery, Moderator
U.S. EPA, MERL-Cincinnati

1. Endel Sepp
   California State Dept. of Health
2. **Richard J. Hansen**
   California Dept. of Fish & Game
3. James H. Reynolds
   Utah State University
4. Henry Gan
   Los Angeles County Sanitation District
5. George White
   Consultant, San Francisco
6. Karl Longley
   U.S. Army

MR. SAULYS, *EPA, Chicago, Illinois, Region Five:* In the beginning, there was some mention about the design factors for chlorination systems. I was particularly interested in the data that were presented on the lagoon systems. I wonder if anybody has investigated more fully the use of alternative forms of chlorine besides gaseous, as a means of optimizing or controlling chlorine feed, particularly, for small systems, say a hundred thousand gallons per day.

MR. WHITE: A hundred thousand gallons per day . . . you could use hypochlorite, but, do not forget, if you use hypochlorite, which would be versus, say 150 pound cylinders, I believe you are talking about three and half to four times the cost of chlorine gas. The installation itself is a standoff. Diaphragm pumps are legion. You can use the same automatic controls for diaphragm pumps. You can have a residual signal plus a flow signal, the same as you can for chlorination equipment. The capital cost is about the same, but your chemical cost would be about three to four times the quantity that you would buy for an 0.1 MGD plant.

MR. SAULYS: We did not find, by the way, any reference in the literature, other than for gaseous chlorine. The only advantage I could see to have the hypochlorite, would be from an operator safety point of view, especially if you have a very inexperienced operator and you want to protect him, and if you are not too worried about cost. Otherwise, I would agree with what has been said.

MR. WHITE: First of all, when using a hypochlorinator, you have to have power, but you do not have to have a booster pump to operate the injector, as you do in the gaseous chlorine. There are some little minor differences in that, but the biggest difference comes when you are treating small water supplies. This is a different ball game. Hypochlorite is very effective and very interesting. The other thing is that in any chlorinator installation that small, where you are using 150 pound cylinders, and because the amount of gas withdrawn is at such a slow rate, the ambient temperature change . . . diurnal temperature change, affects the way you build the house for a gas chlorinator. It has to be better insulated. You do not have to worry about anything except freezing, insofar as a hypochlorinator is concerned. I think that about covers it.

MR. OPATKEN: This one is for Endel Sepp. I thought that, in your Paper about the first plant you tested, you said your effluent had a BOD of about 30, and, in the second plant, you had BOD's of approximately five. Then, I thought you said the applied dose was the same in both cases. And then, I thought I heard you say that the residual for the second case, or the good quality, was about 1.5 to 2 mg/l. And, for the poor quality, or the thirty BOD, it was about 5 or 6 mg/l. Is that correct?

MR. ENDEL SEPP: I do not know. Let me look it up. For the poor quality plant which was the pilot plant, the residuals were 5 to 6.5 mg/l.

MR. OPATKEN: And the dose is still 10 to 11?

MR. SEPP: Yes.

MR. OPATKEN: Then, for the good quality wastewater, your residual is down, but your dose is still the same?

MR. SEPP: They were slightly less. I think the residuals were lower than that. I do not have the data here.

MR. OPATKEN: Can somebody explain that one from the board there?

MR. SEPP: One reason was that at San Pablo, the effluent was nitrified, and there probably was breakpoint chlorination taking place. So, we had a good effect, but a lower residual.

MR. WHITE: I might just add something, or rather an anomaly. We have found, in some plants, where they inadvertently lose the ammonia nitrogen, so that the ammonia nitrogen may drop off to zero, that we do not find any free chlorine, and the disinfection system goes to hell in a hurry. And, that by artificially feeding ammonia nitrogen, we can bring back the disinfection efficiency. The only thing that we think happens is that there are some organic nitrogen compounds in these one or two particular sewages that react very rapidly with free chlorine. Now, this is something that is not generally talked about, because the broad picture has always been that it takes a much longer time for free chlorine to react with organic nitrogen. And, this is shown up in the tail away of the germicidal efficiency of chloramine residuals in sewages. So, every once in a while, we get one of these anomalies that is just absolutely impossible to explain.

DR. LONGLEY: Extending on George White's remarks just a little bit, it has generally been thought that in sewage, when we have good mixing, the initial high rate of disinfection we see is due to the free residual. And, Dr. Walter, who is here, did some work, about ten or twelve years ago, at Hopkins, which was attributed to the free residual. I have noticed in work by Stinquist and Kaufman, and I think by Mr. White here, that they claim that the initial high rate of disinfection that you get in a wastewater is not due to a free residual, because they say, it goes so quickly, but is due to what they call a breaking residual. Now, my chemist friends tell me that these equations we see showing the chloramine reaction do not really describe the pathway very well from a mechanistic standpoint, and probably may have a number of intermediates there. And, what the virucidal species is, as the case may be, is something which we may not even have our finger on, at this point in time. Now there are some who, like myself, still believe that probably what we are seeing is a free residual kill, even though it is very transitory in the case where you have good mixing. I do not know if you will subscribe to that or not, George.

MR. WHITE: Well, I give up on that subject, because I am still waiting for Robert Jolley to use that wonderful mechanism that he has down there, that will tell us, precisely in milliseconds, how long it takes free chlorine to combine with the ammonia nitrogen in the sewage. That will settle that part of the argument.

DR. REYNOLDS: I might throw something else into the mix. The curves that we had, showing coliform removal versus total chlorine residual all went through zero, and that is because they had a forced zero intercept. We originally ran those equations without a forced zero intercept, and we got negative intercepts ranging from a -0.1 to a -1.3, which says at zero chlorine residual, we were getting as high as maybe fifty percent kill, if you look at those correctly. Well, obviously, at least from a theoretical point of view, that cannot happen, unless you consider what has just been said. There is an instantaneous kill before that chlorine is taken up, as chlorine demand, or there is a species of disinfectant in there that we are not measuring.

MR. JIM HAGEN, *EPA, Region Three:* Dr. Reynolds, when you ran your tests, what was the suspended solids levels that you encountered in your unfiltered effluent?

DR. REYNOLDS: The suspended solids level? They ranged all the way from a low of 10 mg clear up to a high of 60, 70, and 80 mg/l.

MR. HAGEN: As a result of some regulations that were issued last October, the state of Maryland is issuing permits with levels at ninety mg/l suspended solids. Do you see any difficulties with the averages that high, in terms of adequate disinfection?

DR. REYNOLDS: On lagoon effluent?

MR. HAGEN: Yes, lagoon effluent.

DR. REYNOLDS: I do not see any problems of achieving a given level of disinfection, with suspended solids that high, in a lagoon effluent, because we are talking, primarily, of suspended solids being algae or possibly *Daphnia*.

MR. HAGEN: What about subsequent release of soluble BOD? You stated that you did not find it at your levels.

DR. REYNOLDS: Our experience was that as long as we did not overchlorinate, and especially, get anywhere near breakpoint, we did not observe disruption of the algal cell wall, or in other words, an increase in soluble COD in our work. And, as long as we maintained fairly low residuals, with reasonable contact times, we did not see it in our work. Now, there is some work out, and I will hasten to add, by Echelberger, in particular, which shows that you do have that happen. We were able to duplicate that, by the way, to a certain extent, in our laboratory, under laboratory controlled conditions where we used heavy chlorine concentrations, but we were not able to duplicate it in the field under normal conditions, unless we went to breakpoint.

MR. HAGEN: Well, he had a paper back in 1971, I thought, that demonstrated that phenomenon. But, you were unable to duplicate that, or you think it was just because his high chlorine levels were . . .

DR. REYNOLDS: Yes, if you will look at the chlorine concentrations he had in that paper, you will see that they are pretty substantial. And, in normal practice, our feeling is, you do not need to be that high, and as a result, you do not have that problem.

MR. HAGEN: There were masking effects and you felt your indicators were sufficient and you did not go beyond that?

DR. REYNOLDS: Yes, we felt that it was not masking the soluble COD, if that was what you were suggesting.

MR. HAGEN: The next question is directed to Mr. Gan. Would you see any problems in dechlorinating an effluent with high suspended solids levels?

MR. GAN: I cannot answer that, because with the effluent we worked on, the suspended solids were usually about 5 or 10 mg/l. None of our experiments had high suspended solids, so I would not know what the response from dechlorination would be.

MR. HAGEN: Would anybody have a feeling on that? See, the problem now is that we have been requested to run power plant work and I just wondered if it would be necessary to do that.

MR. WHITE: Well, personally, I do not think you are going to have any problems. But, I would like to point out the original work that Collins did in which he developed the mathematical model, which I find very helpful in designing chlorination systems providing I know that initial coliform count. He found no variation to the model, up to suspended solids as high as, I think, 120 or 140 mg/l. So, I personally would not be worried about 90 mg/l in a well oxidized effluent, which pond effluent is, or a well oxidized secondary effluent. When you start talking about primary effluent, I have backed off of that, and I claim that primary effluents cannot be disinfected as we think of disinfection in California.

DR. REYNOLDS: I would probably take a little more conservative response to that. I have had some experience, over the last year, with using sulfur dioxide as a disinfectant, as opposed to dechlorination. And, we are trying to work out the specific mechanism of the disinfectant relationship. But, it looks like it could possibly be an oxidation-reduction reaction, which would be very similar in the reaction, chemically, of chlorine in water. If that is the case, and depending on the relative strength of that oxidation-reduction reaction, it is conceivable to me that you might disrupt those algal cell walls. I say, conceivable. I am not sure that I would really buy it in practice, because I think you would have to look at the oxidation-reduction potential between the two to really say whether or not it happened. In general, the amount of SO_2 you add for dechlorination is substantially less than what you would add for, say, disinfection. But, I think it bears some looking at. I would not just throw it out the window because it does not sound right on the surface.

MR. HAGEN: Well, let me ask you a follow-up, perhaps. I guess what I am fishing for is a recommendation as to how we should proceed with the situation. Are you saying that, perhaps, given the dechlorination situation, that we should investigate that further on site before allowing them to construct?

DR. REYNOLDS: Yes, I think it bears some looking at. That is, to completely dismiss it at this point, I think is a little iffy. Now, whether you actually have to run pilot plant work to verify that, I am not certain. I think you could look at the oxidation-reduction potentials between the two and probably do it on a lab-scale basis, as opposed to a pilot-scale basis, to come up with the answer. My first reaction is the same as Mr. White's down here; there is probably not a problem. But, by the same token, I can see where there is a potential there.

MR. HAGEN: Fine, thank you very much.

MR. WHITE: Before I forget it, one of the interesting characteristics of sulfur dioxide, which is different from chlorine, and which simplifies the design, is that you can put sulfur dioxide ahead of the lip of a weir and get wonderful mixing right over in the nap of the weir. You cannot do that with chlorine, because you will start getting aerating. The chloramines will aerate out and get some fuming. But, in that way, chlorine dioxide is easier to handle than chlorine.

MR. VENOSA: I have a question for Henry Gan. In the state of California, when you are required to meet 2.2 total coliform, 23 total coliform, and so forth, are these samples taken at end of pipe? Based upon what Henry has presented this morning that ten minutes after dechlorination, a one- to two-log increase in total coliforms occurs, where must the State of California's compliance standards be met? At end of pipe or in stream? Before or after dechlorination?

MR. GAN: To meet the disinfection standard, most of the treatment facilities have to collect the bacteriological sample before dechlorination. And, in most cases, this is done with the state's approval. Now, for

Los Angeles, the 2.2 standard is for some part in the treatment scheme, so the state really does not require that we sample after dechlorination, but in the treatment scheme. So, we were able to meet that standard, in spite of the coliform aftergrowth. Does that answer your question?

MR. WHITE: It also is very critical where you sample it. You learn these things the hard way. When you have an effluent that is discharging into a tidal stream, you do not put the sample point where it can get the back flow of the tidal stream, because then you get the aftergrowth, after dechlorination. So, you want to be in an appropriate place upstream from dechlorination, which does not get that back diffusion of the dechlorinated effluent.

MR. VENOSA: It seems to me that the purpose of a coliform standard is to control discharge of microorganisms into a stream. And, you have an aftergrowth problem based strictly upon where you take your sample. It seems to me somewhat of a cop-out to allow a sample to be taken prior to the dechlorination, just to enable compliance with the standard.

DR. REYNOLDS: Coliforms, as everybody knows, are only an indicator. Do we know for certain that, just because coliforms are regrowing, the actual pathogenic organisms are occurring in the regrowth? And also, I was just going to mention, with some of Henry's data, especially taken along the canal where it is unlined, some of those coliforms are probably coming from leaching out of the soil, as opposed to actual regrowth in the canal. But, I really do not know. Has there been any work done to indicate that when coliforms regrow the pathogens are regrowing?

DR. LONGLEY: I think work done by Chambers and a number of others, over the years, has shown that the pathogens, for the most part, do not regrow. And, if, in fact, we can consider coliforms a suitable indicator, I know that George has some comments on that too . . . but, if they are, in fact, a suitable indicator and we can show that at some point in the system we have suitable indicator kill, we, therefore, assume that we have killed the pathogens and they are not regrowing. If the coliforms regrow downstream this is maybe indicative of other problems, but it is certainly not a problem of having pathogens in your downstream.

MR. PETER DeSTEFANO, *Thomas M. Riddick and Associates:* I think you may have answered the question already. But, I was a bit confused with this regrowth problem. If the effluent had a residual of five parts per million, there probably was not anything still alive. Are we talking about coliform regrowth or recontamination from some sort of infusion from the plant or aerosol or some other way? Mr. Gan?

MR. GAN: In my presentation of data, I said it is an aftergrowth, which means that there is a possibility that the coliform is in the chlorinated effluent, but only to a small amount. But, then after dechlorination, you are creating an enviroment where you have rapid aftergrowth of the coliform, in the floc material.

MR. WHITE: Maybe I could summarize it by reading what I wrote about it in my new book.

It is under the heading of The Regrowth Phenomenon. It is easy to find because it is in the index. "Numerous studies have demonstrated the regrowth phenomenon of coliform and fecal coliform organisms after disinfection. This may be due to the destruction of bacterial predators and may depend upon the presence of certain nutrients in the wastewater or the receiving waters. It has been observed both in wastewaters and in receiving waters downstream from a disinfected sewage discharge. It is the judgment of authorities, such as Geldreich, that pathogenic bacteria, like *Salmonella* and *Shigella*, of the same family as the coliform group, also regrow. If the disinfection process has a comparable effectiveness against such pathogens as with coliforms, a disinfection requirement, resulting in reduction of numerous coliform bacteria, down to a level of 23, or 230 per 100 ml, would virtually assure the absence of pathogenic bacteria in sewage in the smallest numbers, etc." But, this is the view of the California State Department of Health, that the pathogens are knocked out.

MR. VENOSA: I would like to make one comment on that, George. I would have to respectfully disagree on one thing. You are talking about nutrients and absence of predators and so forth, and Henry presented evidence that regrowth occurs within ten minutes. There is no known bacterium on earth that is capable of dividing in ten minutes. It cannot possibly be "growth". That is my opinion. I do not think it can be growth. I like the term "recontamination" that was just suggested from the audience.

MR. WHITE: I understand what you are saying. But, do not forget, when you have been dechlorinating for a long time, you have an entirely different biomass downstream from the dechlorination point that you do upstream. This makes a big difference.

MR. BOB MASTERSON, *Tufts University:* Not to add another term to it, but at Tufts we have experienced maybe what you would call reactivation. And, in a tertiary effluent we had what we thought was regrowth, but, on the same sample, we had a die-off of the unchlorinated one. Therefore, the substrate was insufficient to support reproduction. But, the injured organisms, we felt, were coming back and surviving the actual chlorination. The levels were not nearly as high as five, but there were around one.

MR. READ WARRINER, *CH$_2$M Hill*: I would like to ask about optimization, in connection with the first two papers this morning. I gather it is defined as rapid initial mixing, thirty minute contact, in a residual control system. Could you tell me the basis for each or any of these three and possibly also, the name of the proprietry equipment that was employed in the pilot plant?

MR. SEPP: The pilot plant has a static mixer . . . a tubular mixer. It is simply a T in a three-inch pipe. It has a Reynolds number of 34,000. And, we have just installed a 1.5 inch T. We wanted to see whether that makes any difference. I do not know if it does or not. And, mixing is accomplished within two seconds, which is pretty fast. The control system consists of a residual analyzer and a chlorinator. The residual analyzer has an automatic closed loop control. It is based on direct residual control. The flow is constant, so therefore, we do not need any flow proportional control. The contact tanks are long narrow baffled tanks with a length-to-width ratio of 130 to 1 and 1 hour contact. Design was based on what was thought to be good practice. I understand many plants have the same kind of a mixer . . . they have a diffuser in a pipe. But, you have to provide turbulent flow in that pipe, otherwise it will not do much good. Does this answer your questions?

DR. LONGLEY: I would like to make a comment. In a very general sort of way, Reynold's number is, of course, as far as mixing is concerned for chlorine, an indicator of whether or not you have a good or poor mixing. But, from a design standpoint, we have found in some work we presented at ACS last March, and in some work we will be presenting in the future, that Reynold's number, in fact, cannot be correlated to the amount of inactivation you get, given a certain sewage and a certain amount of chlorine dose to that sewage. In truth, the Reynold's number does not describe such factors as the chemical reaction rate. And, most of these systems are either reaction rate limited or diffusion limited. I tend to believe that in practically every system we have, we still are diffusion limited, particularly in the dirty systems. The microorganisms are going to have a certain amount of organics associated with them that are going to compete with the chlorine even if you remove the solids. The problem is trying to get the chlorine to the active sight on the organism, whatever that may be, whatever type of mechanism there is, prior to the conversion of the chlorine to a less active biocidal species. So, to be able to describe this, we need some other descriptor than Reynold's number . . . something which can express time, concentration gradients, and things such as this. Now, I appreciated some of the work that was presented on the ponds this morning, simply because it attempted to get away from the black box type of design. Hopefully, some day, when we are talking about wastewater disinfection, we can describe it in some sort of a formalistic form that will give us a handle that we can use when we start to design these systems, other than just black boxing them.

MR. WHITE: I have long ago abandoned the idea of trying to use the Reynold's number described by Karl. What I have been using lately is based on some work of the County Sanitation Districts of Los Angeles, with whom Henry Gan is associated, and I also had the opportunity of working on a couple of jobs with some consultants. We tried to pick an arbitrary number of the mean velocity gradient G. LA County used 500. I do not know whether they picked that number, but that is the way it came out . . . 500. Obviously, working down to 2.2 MPN, they got good results in their work in the plant. I would raise it to 1000. If you needed 23.2 or 2.2 MPN/100 ml, if possible, I would design my mixer so that the mean velocity gradient is 1000.

DR. JOHNSON: On the other side of the coin concerning the residual control, of course, we know the detention time, residual concentration, turbulence, all are important factors in the disinfection process. The literature is replete with data that show that wastewater disinfection control through residual is a little iffy. I wonder whether or not this might be due to a couple of different things. One is, the residual, of course, changes quite a bit with time. You put in ten parts and maybe it comes down to five, in the first second of rapid demand. The residual concentration is decreasing all the way through the contact tank. Usually, we just measure the residual at the end . . . thirty minutes or so, and there is a lot of change in concentration. You can measure the residual all the way through, or you can measure the residual after two minutes, in order to do closed loop control. But, most plants are manual control plants, and they control on the basis of a thirty minute residual. An operator must crank the control every thirty minutes.

MR. WHITE: That is known as a poor control set up.

DR. JOHNSON: I am talking about standard practice today.

MR. WHITE: It is not in California.

DR. JOHNSON: California is an unusual place. Anybody that disinfects down to 2.2 MPN/100 ml has to be unusual, right? So, the residual changes as a function of time, not just with respect to the concentration, but also with respect to type of residual. I have seen, in my data, a change from primarily a monochloramine type residual, in the initial part of the chlorination process, to primarily a dichloramine residual, at the end of the disinfection process, after thirty minutes. I have forgotten whose data it was, but I found it interesting that they get any additional disinfection from thirty minutes to sixty minutes. Whose data was that?

MR. VENOSA: Jim Reynolds'.

DR. JOHNSON: Yes. I wonder if that might be due to the changing character of the residual. I wonder whether or not we should be controlling more wastewater plants on the basis of the monochloramine residual which tends to be more inorganic chloramine, instead of the total residual which has the dichloramine and probably a lot of organics in it. I just throw that out as a kind of point of discussion to see what you all think of that.

DR. REYNOLDS: I think that when you start comparing coliform removal efficiency, as a function of chlorine residual, after whatever contact time you speak about, whether it be eighteen min., thirty-five min., an hour, or whatever the case may be, you have to look upstream at how the chlorine is injected, the degree of mixing, and the actual contact time. Whether you really are getting that kind of contact time that is presented with the data. I think that has a great bearing on this iffyness that you talked about in coliform removal efficiency as being controlled by chlorine residual downstream, because all of those factors go into that residual. I think in our case there are probably two things which contributed to the fact that there was no more removal, or no different removal between thirty-five and fifty minutes. We had very intense mixing and good contact with the disinfectant early in the process, and most of the organisms were affected by the thirty minute contact time. We just did not see any additional kill after thirty-five minutes. We were also getting down to some very low coliform concentrations by that time.

MR. WHITE: I think the important thing, in working on contact times, is t_i, which Endel Sepp referred to, and that is the initial appearance of dye. So, what I have been telling my clients, when they design a contact chamber, is that they have in the contract that there has to be a dye test made for the contact tank at three different flows, provided they have a reasonable amount of wastewater to put through. Usually, in new plants, you cannot vary the flows too much. So, you have three flows, and you do this two or three times and you get some kind of a mean value. You do not worry about the peak of the curve, where the dye is, you worry about the t_i. That is the first appearance of the dye, and that is the thing that you plug into Collins' formula for your kill.

Now, going back to my shaking my head at Don Johnson while he was telling about the residual. The only good control system is one that takes the two or hree minute chlorine contact time for the control of the chlorination equipment. Then, you can plot the die-away. You are going to have a residual analyzer at the end of the system anyway for dechlorination, so you can use that other analyzer to come up with the residual concentration time, at the end of the contact chamber, which is t_i, to plug into the Collins' formula. Collins' formula is close enough to design the capacity of the equipment. This is the whole point. That is what we ought to do with contact chambers. We do know that the germicidal efficiency tails away in thirty minutes. We think that is because the monochloramine and the dichloramine start to hydrolyze and form organic chloramines which are less germicidal. Nobody has really proved this, but they have come pretty close to it. I agree with Dr. Reynolds that beyond thirty or thirty-five minutes you are not improving coliform kill. But, if you want virus inactivation, Los Angeles County has proved that at the end of about fifty or sixty minutes, chloramines catch up with free chlorine and with ozone. So, there are two different ball games. You are going to kill viruses and you are going to kill coliforms.

DR. JOHNSON: What about the difference in controlling the monochloramine compared to controlling the total chlorine?

MR. WHITE: We have enough trouble with controlling total chlorine to get into that one. But, I am working on a couple of things with cells where we do not go through the classical way of measuring iodine release in the total residual, which comes closer to what you are saying. It looks pretty good.

DR. LONGLEY: Just a little more on developing one point that George made. Some work by Stinquist and Kaufman at the University of California a number of years ago, which we more or less duplicated, showed that, if you had just fair initial mixing, the initial coliform kill was only moderate. They were injecting their chlorine in a pipe with one outlet and they were using a grid for their good mixing. With very good mixing, they achieved a high initial kill of coliform, much more so than they did with just the center point injection. However, after thirty minutes both were the same. In the second system a very high initial rate of kill was achieved and then very little decay thereafter. In the other system, a more or less uniform kill occurred throughout the thirty minutes, thereby achieving the same final product. We have seen this same phenomenon in a pretty well mixed system and in an extremely well mixed system. In an extremely well mixed system, you are just shoving energy into it to get the initial mixing, really to no added benefit at thirty minutes.

MR. WHITE: But, was your contact chamber completely plug flow?

DR. LONGLEY: Yes, the contact chamber was completely plug flow.

MR. WHITE: Okay, that is the difference.

MR. SAULYS: In the midwest, one of the battles that the biologists have been fighting is trying to instigate total residual control. As far as I am aware, in most of the counties using free chlorine residual control, where the state engineers are progressive, they limit chlorine residual at the end of the discharge from the contact chamber to between 0.2 and 0.75 parts per million. Total residuals could be as high as 10 or more parts per million.

MR. WHITE: I am confused. Did I hear you say something about 0.2 mg/l free chlorine residual?

MR. SAULYS: That is correct.

MR. WHITE: There isn't any 0.2 mg/l free chlorine residual in wastewater effluent, unless you are nitrifying.

MR. SAULYS: In the NPDES permits issued by the state of Illinois, many require sewage treatment plants to discharge a free chlorine residual from the contact chamber into the river or receiving stream between 0.2 and 0.75 mg/l free chlorine residual, after a half hour of contact time as designed.

MR. WHITE: You would have to breakpoint it and then some.

MR. SAULYS: That is why we end up with pretty high totals.

MR. WHITE: Even if you breakpoint it, all your residual would not be free chlorine. You would have some dichloramine and maybe some monochloramine present. That kills the fish, too. What are you going to do with that?

MR. SAULYS: I agree, but this is the problem that the biologists are facing out in the midwest, where the engineers, in the interest of public health, are requiring high chlorine residuals, particularly, free chlorine residuals, to insure a good disinfection, to insure kill, with the resultant effect that some of our streams that are being cleaned up are pretty damn sterile, in terms of aquatic life.

MR. WHITE: It is absolute folly to try to specify any kind of a residual and say that this is a disinfected sewage. The only way to be sure is with coliform testing. Are any of these engineers, from these places that you speak of, here? They are going down the wrong track, in my mind.

MR. SAULYS: Maybe next time there is a standards hearing in Region 5, I will take some efforts to invite you down and speak to the boards.

MR. WHITE: Okay. I will start a riot, maybe.

MR. VENOSA: I thought I was recently informed that the State of Illinois had done away with their coliform standards altogether. Isn't that true?

MR. SAULYS: It may be another year or so before they decide to issue a standard. I am not aware that they have changed it. They have held a number of hearings. They have tried to do away, at least, with year around chlorination. They received a mixed reception from USEPA, where some people came out in support of them, others warned them of public health menaces. The state is sitting on the whole question right now, because, potentially, it is a hot political issue. The issue had come up, for example, in the state of Wisconsin. It came up before the Department of Natural Resources Board. One of the members stood up and said that one of his relatives had contracted polio in the past and he did not want it on his conscience that anybody would ever get polio again because we are not chlorinating year around, and voted down the proposal to seasonal chlorination. This is the type of situation, at least, that the states in the midwest are faced with right now. Some four or five years ago there was seasonal chlorination. There was judicious use of chlorine. The federal EPA came in and imposed year around chlorination through the grants system . . . through the enforcement system. Now, when the USEPA is trying to back down, trying to balance some of their aquatic concerns against public health concerns, we are again receiving resistance because everybody is used to chlorinating and they would rather be safe than sorry.

MR. WHITE: Well, 0.2 mg/l free chlorine in half an hour is not going to kill any polio.

MR. CAL SAWYER, *Dewberry, Nealon and Davis in northern Virginia:* I have a couple of questions. The first to Mr. Hansen. When you talk about lethal concentrations and mixing 53% dilution of your chlorinated wastewater with your control water, did you work with your unchlorinated wastewater, same samples unchlorinated, mixing with 53% and exposing the fish to this?

MR. HANSEN: We had a hundred percent of the unchlorinated water and all test fish have survived.

MR. SAWYER: So, that the fifty-three percent was the . . .

MR. HANSEN: Fifty-three percent is the LC50. That was that point where half the population was calculated to have died in the test.

MR. SAWYER: They all survived in a hundred percent effluent?

MR. HANSEN: Yes.

MR. SAWYER: Next question to Mr. Gan. You indicated in your results that dissolved oxygen adjustment and pH adjustment were not necessary on most of these plants because of the high alkalinity content. What is the lower level of alkalinity where you would need pH adjustment, if you were using SO_2 dechlorination?

MR. GAN: I cannot answer that question because what we did is just monitor the plant effluent, and all our treatment plants have an alkalinity of 200 mg/l. So, I do not know what the response would be down at lower alkalinity levels.

MR. SAWYER: Once you get about 200 mg/l of alkalinity, does your SO_2 to chlorine residual ratio go up above one-to-one?

MR. GAN: Yes. It will go up about two-to-one.

MR. SAWYER: Maybe if you are below a hundred mg/l of alkalinity in your effluent, you might be thinking of pH adjustment?

MR. GAN: It is a good possibility, yes.

MR. SAWYER: How about with dissolved oxygen?

MR. GAN: There was natural aeration going on through the treatment systems, and right in our SO_2 mixing chamber there was turbulence, mixing the SO_2 in the solution. For that reason, we never had any problem with having to reaerate our water.

MR. SAWYER: If you are using aeration for mixing the contact chamber itself, probably there would be no need for post aeration?

MR. GAN: Yes. It depends on how much SO_2. In our Pomona plant we do not even have mixing. We just add the SO_2 right at the out-fall. It is just the turbulence at the out-fall that mixes the SO_2 hydraulically with the solution.

MR. SAWYER: Do you think in most cases that you need a rapid mixer to make SO_2 effective?

MR. GAN: Definitely, yes.

MR. RON SOLTIS, *Washington Suburban Sanitary Commission:* I am mainly concerned with operation of existing wastewater facilities. I have heard a lot today on monitoring and taking chlorine residuals, and I have seen some of the graphs up here that have zero chlorine residual as points on the graphs. At the plants that we have to operate under current discharge permits, we are running into a problem with chlorine residuals approaching 0.02 total chlorine residual in the effluent. There was reference earlier to Blue Plains which has gone down to 3 parts per billion free, in the effluent. The question that I have (and I am wondering what EPA is doing or not doing), is where do you all stand on research into how are we, the people in the field, going to monitor these wonderful discarge limitations so that we can say, yes, EPA and water resources administration, we have noncomplied with the permit, although we did not kill anything, but we do not know what the heck we did. I really have a problem with it, and with operators who use amperometric titrators. The amperometric titrator is only accurate down to what the limit is, so essentially you do not know whether you have ever gone below the limit. You are either at noncompliance level or you do not know where you are. Is any of the research that EPA is funding going toward operation, rather than whether we are going to kill anything or not? How are we going to find out what we are doing?

MR. BEN BUONOMO, *Delta Scientific Envirotech:* I was pleased to hear that question because no matter what we do in a laboratory, we are using portable instruments. We are measuring discrete samples. In the real world, all of our fine measurements and the theory and the models that we develop from these portable instrumentation, measuring discrete samples in a clean environment, does not hold true when you are on continuous on-line monitoring, twenty-four hours a day, seven days a week. We recognize this problem of measuring very, very low chlorine residuals. With utilities discharging into the river waters, in the regions we have worked, 0.25 mg/l is the maximum. When they are discharging, as with condensor cooling water into an ocean, it is 0.5 parts per million maximum. EPA now, in Cincinnati, under Tony Mentink's quality assurance laboratory, is evaluating a polarographic membrane technique for monitoring chlorine residuals. We have one now working in a laboratory, 0.25 parts per million full-scale. Currently, we are working on one that was the zero point off the end of the scale, not on it directly, so we can even measure negative when we are doing sulfur dioxide, to see how much have we dechlorinated. There is some work being done by the EPA with us.

MR. VENOSA: Excuse me, how well does your membrane electrode work in the wastewater environment?

MR. BUONOMO: It is an iodine release from iodide, so we measure total release.

MR. VENOSA: Yes, but I mean as far as gumming up with solids and so forth?

MR. BUONOMO: Thank you for that question. Instead of exposing naked electrodes as conventionally has been done for the last ten or fifteen years in an amperometric cell where your electrodes are exposed completely to the effluent, we isolate the electrodes from the process liquid by a membrane. It is constantly enclosed with a clean electrolyte and the membrane is blocking the effluent. Through the membrane, the specific chlorine species are coming through that we monitor.

MR. SOLTIS: I have one other question, again, concerning operation. We have several . . . actually two plants now, that have upflow carbon columns that are supposed to have been designed for dechlorination They were designed with a free residual of three going into the carbon column, and the effluent coming out was supposed to be 0.05 or less total chlorine. These were supposed to last for the life of these plants which were interim. We found out that after approximately three to four months, in order to stay within the permit limitation, we could no longer put in three mg/l free chlorine. We had to go down to between 0.5 and 1.0 free chlorine in order to stay at the 0.05 total residual out in the effluent. I previously asked if California had any systems that were actually on-line, and the answer, I believe, was no, at that time. I ask the panel or anyone in the audience, if there are any full-scale facilities that are actually using either up-flow carbon columns or any other type of carbon column, as a means of dechlorination?

MR. WHITE: I believe there is one that is either in the process of design or construction, and that is at Vallejo, California. But, it sounds to me like you have a whole lot of problems. The easiest way to do dechlorination with SO_2 is either by the biased residual that Mr. Gan had a little schematic on, or to just put in a slight excess of SO_2 on the feed forward basis, and intermittently sample the dechlorinated sample on your control analyzer, through a timer, five minutes out of the hour. Those two work very well.

MR. SOLTIS: What we have gone to is an SO_2 system as a backup. Because of the investment in the carbon and the facilities for storing the carbon, or actually running the flow through them, we utilize the carbon to remove the maximum amount of chlorine possible and then we trim it with sulfur dioxide. What I am looking for is help in trying to find out (which we have not been able to get an answer for yet) what has happened that the carbon has dechlorinated in both cases, satisfactorily for three to four months, and then over almost a one to two week period, it was almost a straight line drop from 3 mg/l free down to 1.0 or 0.5 mg/l.

MR. WHITE: Well, is this regenerated carbon or is this just . . .

MR. SOLTIS: No, this was designed to last the life of the plants, which was . . .

MR. WHITE: Eighteen months?

MR. SOLTIS: No, it was two years. And, in both cases . . . one is a 200,000 gal/day flow and the other one is 2 million. And, in both cases, it just did not last longer than the three . . .

MR. WHITE: All right, if you have a backup dechlorination for SO_2, then the easiest thing to do is to come down to a half a milligram per liter total, which is easy to do with conventional chlorination equipment, and use that as a feedback for your control, and then dechlorinate the other half. But, do you do this? Is this what you are doing, and you mean that the carbon is petering out?

MR. SOLTIS: It has petered out. We have a zinger in here, in that we have to maintain 3 mg/l free chlorine residual prior to dechlorination. At the end of one hour contact time we have to have a 3 mg/l free chlorine residual for virus inactivation.

MR. WHITE: 3 mg/l free chlorine?

MR. SOLTIS: It is one of the requirements, not of EPA or the state of Maryland, but it is a resolution of the county that we have.

MR. WHITE: I see.

MR. SOLTIS: They are trying to put a virus limitation on us, and one of the requirements, like I say, is a 3 mg/l free chlorine residual, at the end of one hour contact time, prior to dechlorination.

MR. WHITE: What kind of coliform counts are you getting before dechlorination, by the carbon?

MR. SOLTIS: In most cases, the coliform is less than three.

MR. WHITE: What do you do to take care of the carbon and keep it from fouling?

MR. SOLTIS: The plants are all designed with either a chemical addition, for tertiary treatment, and filtration prior to chlorination . . . we have to go through filters, so the effluents, in most cases, have turbidities, for instance, in the 0.6 range and suspended solids less than 5.

MR. WHITE: Yes, but don't you get solids fouling in the carbon?

MR. SOLTIS: So far, in these two particular facilities, we have not noticed any.

DR. ARCHIBALD HILL, *Louisiana Tech University:* I have a question for Henry Gan. I was interested in your recontamination after dechlorination. You had ten minute and two hour samples. I was wondering if these were estimated from points on your contact chamber, and if so, could part of the recontamination have been due to back mixing effects?

MR. GAN: The time is based on points in the contact chamber. The recontamination, we are sure, is not due to back mixing, as I showed in one of the figures. Initially, we do not get any recontamination, or did not get any recontamination right after cleaning of the tank. The recontamination occurred after a few days operation. Another thing I want to add is, we are certain there is recontamination because we have collected grab samples in dechlorination experiments in the lab, and we found no coliform increase several hours later. But, we did find it in the dechlorination chamber.

DR. GILBERT GORDON, *Miami University:* Later today and tomorrow, we will hear about chlorine dioxide and ozone, primarily, because of some concern over removing haloforms in general. Are any studies being carried out on the chlorination test to see what kind of reduction or what kind of chloroform and any haloform residuals are formed as a function of the various ways in which mixing and chlorination are carried out?

MR. WHITE: It is well documented that the chloramines contribute very little to the THM's. They contribute other organics. The biggest concern is in potable water because that is where the precursors are.

DR. GORDON: But, if you remove the precursors, perhaps, at one of the . . .

MR. WHITE: Yes, but now you are getting into water treatment.

MR. MIKE WITT, *Wisconsin Department of Natural Resources:* I would like to ask Mr. Venosa, what, if any, research EPA has going, as far as the need for disinfection? The program here is geared a lot toward different types of disinfection. As the gentlemen from Region 5 said before, one of the questions we have in the midwest is whether or not we need it at all in the winter time, since nobody swims in ice. I would like to know what your opinion is of that, and if there is anything that we can get ahold of that might give us more information?

MR. VENOSA: We have a project that we have planned for next year to try to determine, or assess, the significance of antibiotic resistant coliforms in sewage effluents because the increase in the number of antibiotic resistant coliforms could someday, possibly, result in an increase in the number of antibiotic resistant pathogens as well. Maybe that would be one argument in favor of disinfection at the wastewater treatment plant, to control the proliferation of such types of organisms. So, we have a project to address that type of problem. The Health Effects Research Laboratory is conducting some research to establish correlations between coliform numbers and epidemiological transmission of disease in bathing beaches in the salt water areas of New York. I am hoping that they do that in fresh water, too.

MR. WITT: In our work, about a year ago, we tried to make that administrative change in Wisconsin, and go to seasonal disinfection. It was clear at that time that no one could make any correlation either way. We tried to show one side, and other people tried to show the other side. Some of the research, at the University of Wisconsin, has dealt with the probabilities of different disease transmission routes, showing that wastewater disinfection, whether you have it or not, makes little difference. People get diseases from other sources. We also have a person with the Department of Medicine, at the University, doing some research on disease transmission in children swimming in bathing beaches, swimming in pools, or not swimming at all. The children that swim in the pools get sick the most. The thought is that, perhaps, it is because there are so many so close together, and not because they jump in the water.

DR. LONGLEY: I think the problem though is when you try to do any of this type of epidemiology, the numbers that you are incurring are so low that they are lost in the total numbers, and to try to attribute any significance to them becomes a burden, which we really have not found out how to deal with yet. The same problem, of course, holds with cancer caused by THM's. The other point is, if you are going to accept that we might have, in fact, a virus problem, I think you have to realize that viruses can survive for some long periods of time in the environment. If we put them out in the winter time, they might still be there next summer.

DR. REYNOLDS: Another approach to that is looking into actual coliform die-aways in various treatment trains and systems. As I indicated earlier, a well designed lagoon system, with sufficient contact time, will achieve very low coliform numbers in the effluent, with no disinfection. EPA has funded (and those reports, I think, are available) about nine systems nationwide, where the quality of the lagoon effluent was monitored. You can check those systems to determine the need of chlorinating lagoon effluents, if they were properly designed. A lagoon effluent is not an activated sludge or a trickling filter effluent.

MS. BARBARA BROOMFIELD, *Argonne National Lab:* Now, aside from the already stated health effects of chlorinated and nonchlorinated effluents, I was wondering if anyone bothered to assess the effects of nonchlorinated or nondisinfected wastewaters on test organisms or other aquatic organisms? Mr. Hansen mentioned that he observed no death rate from organisms exposed to nondisinfected wastewaters, but did he bother to examine them to notice if there were other physiological changes that might have taken place in these organisms?

MR. HANSEN: No, we did not examine them any further. What we are looking for, and I thought I made it clear, is for acute toxicity. That is all we are looking for in this particular study.

MS. BROOMFIELD: Would there be any studies being done to determine if there were any long-term effects from discharging large amounts of nondisinfected wastewaters into the streams and rivers?

MR. WHITE· Yes, there was a study, a very comprehensive study in San Francisco Bay. To answer your question, very briefly, I think the man's name was Brock, but it was done under the Sanitary Engineering Research Laboratory, University of California, about 1969-70. They found out that chlorinated and dechorinated effluent were less toxic than an unchlorinated effluent. This was on the biomass . . . attached organisms, if you will, that were growing. Before I forget, I want to tell you about this dechlorination problem, and the contact time. I have seem somewhere in print, that the EPA or some other regulatory agency, did not consider an out-fall as a contact chamber. On the contrary, a long out-fall is absolutely the best type of contact chamber that you can make. If you are smart enough when you are designing it, you can wind the out-fall around the plant long enough to get your thirty minutes, and dechlorinate to 0.5 mg/l and then the rest of the out-fall will accumulate enough biomass in there to destroy the remaining residual. This has been done in a couple of places and it is very effective. Of course, Fish and Game might not like that idea, but it works.

MR. VENOSA: I would like to provide some more information for Miss Broomfield. We funded a study with Wyoming, Michigan, (using two study sites, Grandville and Wyoming) where we evaluated the effects of chlorinated, dechlorinated, brominated, and ozonated effluents on fish, both acute and chronic toxicities. If I remember correctly, at least in the Grandville portion of the study, the dechlorinated, ozonated and unchlorinated effluents were nontoxic. The ozonated effluent was not only nontoxic, but actually stimulatory, so that in the chronic exposures the fish actually grew bigger. They laid more eggs and the hatchabilities were higher than the controls. Presumably, this was due to the higher dissolved oxygen content. At Wyoming, Michigan, we repeated the experiments, but the unchlorinated effluent (control) was toxic because half the raw wastewater was of industrial origin, containing cyanides and other toxic compounds, so we really could not tell too much from that one.

MR. GREG SEEGERT, *WAPORA, Inc.:* Before asking my question, I would also like to respond, briefly, to the lady's inquiry. The problem that you typically get into is, there is no such thing as a typical effluent. There are many effluents which are just plain bad news, whether they are chlorinated or not. So, as Al was just mentioning, in Wyoming there was a high industrial input with lots of heavy metal contamination. The effluent was highly toxic whether it was chlorinated or not. In many cases, if you are talking about relatively high quality water systems, there is not much of a problem. It depends almost entirely on the overall quality of the effluent. Chlorine only comprises a portion of the potential toxicity. My question is for Mr. Hansen. You had characterized all your LC50 values, in terms of percent effluent. I was wondering if you had calculated what the total residual chlorine concentrations were for those same LC50 values?

MR. HANSEN: Yes, I have. As I recall, the higher one was 0.60 total residual chlorine. The lower one was, I believe, 0.13, at San Pablo.

MR. SEEGERT: The higher value was for the golden shiner? You said that the golden shiner was considerably more resistant?

MR. HANSEN: No, this was on fathead minnows. At San Pablo, the total ammonia was less than one, and the 96-hour LC50 was 0.60 TRC. Effluents from all the other plants had ammonia concentrations greater than 10 and mean 96-hour LC50 of 0.13 TRC.

MR. SEEGERT: And that was for the fathead minnow?

MR. HANSEN: That was for the fathead minnows, yes.

MR. SEEGERT: What was it for the golden shiner, if you have that information?

MR. HANSEN: I do not recall if I have that.

MR. SEEGERT: As I recall, you did make the point that the fathead minnow was considerably more sensitive.

MR. HANSEN: Yes, that is correct.

MR. SEEGERT: Al probably also has the information, but as I recall, in the study that was done at Grandville and Wyoming, I think, the golden shiner was the most sensitive species that they tested. The 96-hour LC50 values were about 40 parts per billion, and the fathead minnow was approximately two to three times more resistant, which would place the overall toxicity almost an order of magnitude lower than your LC50 values for 96 hours, and also would reverse the relative sensitivity of the two species. Would you respond to that, please?

MR. HANSEN: Well, in the ones that we calculated, we found that they did agree very closely with what we found in the literature.

SECTION 3. CHLORINE DIOXIDE

9.

EFFECTIVENESS OF CHLORINE DIOXIDE AS A WASTEWATER DISINFECTANT

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ABSTRACT

Chlorine dioxide (ClO_2) is evaluated as an alternative to chlorine (Cl_2) for use in the disinfection of municipal wastewater. Bench-scale experiments in three topic areas are discussed. First, two methods for counting bacteria, Membrane Filter and Most Probable Number, are compared. Second, factors affecting the relatively inadequate performance of ClO_2, based on kill of coliform bacteria, are investigated. Third, the two disinfectants are compared according to inactivation of viruses in secondary effluent. The information from these experiments will ultimately be used to develop a concept for a continuous reactor system that will be tested on a pilot scale.

INTRODUCTION

Alternatives to chlorine for the disinfection of wastewater are being sought for several reasons, such as developing a process that is more efficient and that yields an effluent that is environmentally safe. A viable candidate to replace chlorine should meet the following set of criteria. The disinfectant should:

1) be cost effective;
2) provide a measurable residual;
3) be easy and safe to handle; and,
4) produce a minimum of undesirable by-products.

Chlorine dioxide (ClO_2) is being investigated as an alternative under EPA Research Grant R-805426, "Feasibility of Using Chlorine Dioxide in the Disinfection of Municipal Wastewater." Chlorine dioxide has been used successfully in Europe to disinfect water supplies and is used extensively as a bleaching agent in the pulp and paper industry. With respect to the disinfection of wastewater, chlorine dioxide has several exceptional characteristics; it

1) is a strong oxidant over a broad pH range;
2) provides a measurable residual;
3) does not react with ammonia to form less effective chloramines; and,
4) does not react to yield trihalomethanes.

The principal objectives of the feasibility study are to: 1) use these characteristics to develop a concept for a continuous reactor system that will be tested on a pilot scale and used to prepare a preliminary design for full-scale treatment facilities and 2) compare chlorine dioxide and chlorine as wastewater disinfectants on the basis of effectiveness and cost.

This paper will address three topics that describe the behavior of chlorine dioxide in secondary effluent and begin to define those characteristics that will lead to a process design. The three topics, from the first nine months of the study, are:

I. Comparison of MPN and MF Methods of Counting Bacteria.
II. Factors Influencing the Bactericidal Efficiency of Chlorine Dioxide.
III. Virucidal Effectiveness of Chlorine Dioxide.

GENERAL METHODS

MPN and MF tests were done according to *Standard Methods* (1975)(5). MPN media used were lactose broth as a presumptive medium and brilliant green bile broth and EC medium for confirmation. Gelman GN-6 filters were used for the MF tests and M-Endo Agar for total coliforms using the single-step direct technique and M-FC medium for fecal coliforms.

Virus samples were collected, placed on ice, and transported to Dr. Robert Cooper, Sanitary Engineering Research laboratory, University of California at Berkeley, for analyses. Transit time was less than 2 hours and the analyses were begun immediately upon receipt of samples. The animal virus used in the study was Poliovirus I LCS strain that had been grown in Buffalo Green Monkey Kidney cells (BGM). The viruses were harvested from the cells by freezing, thawing, and centrifugation to a titer of approximately 10^7 plaque-forming units per milliliter (PFU/ml). The virus was recovered after experimentation using a standard plaque assay. A coliphage that had been isolated from Palo Alto secondary effluent (the wastewater used throughout the study) was grown on host *Escherichia coli B* to a titer of approximately 10^9 MPN/ml and used in the experiments. The phage was recovered and assayed using a modification of the MPN described by Kott (1966)(2).

Secondary effluent from the Palo Alto Water Pollution Control Plant was used in our experiments. Samples were collected at about 8:00 A.M. each experiment day to minimize problems of variability due to hydraulic surges and to minimize NH_3-N concentrations. The effuent was characterized by measuring the following parameters:

1) total COD
2) alkalinity (by an amperometric titration)
3) pH
4) temperature
5) total filterable residue (dried at 103°C)
6) ammonia as nitrogen

All parameters were measured singly except for the total filterable residue (total suspended solids) which was done in duplicate. All tests except for ammonia nitrogen were done according to methods prescribed in *Standard Methods*. Ammonia nitrogen was measured using an ammonia electrode. Previously, results using the ammonia electrode and those of nesslerization (a procedure which is prescribed in *Standard Methods*) had been compared. A t-test based on 13 paired values in the range of 0.5 to 30 mg NH_3-N per liter showed no significant difference between the ammonia electrode and the determination by distillation and nesslerization.

The oxidant chemistry consisted primarily of chlorine dioxide and chlorine generation and standardization, and residuals measurement. Chlorine dioxide was generated using the sodium chlorite acid activation technique described in *Standard Methods*. A suggested chemical reaction for this procedure is:

$$4H^+ + 4ClO_2 \rightarrow Cl^- + 2ClO_2 + ClO_3^- + 2H^+ + H_2O.$$

The schematic drawing of the reactor is shown in Figure 1. Chlorine solutions were made by bubbling chlorine gas through distilled water. Both chlorine dioxide and chlorine solutions were standardized at the beginning of every experiment using the iodometric method prescribed in *Standard Methods*. Sodium thiosulfate was placed in the sampling flasks to reduce chlorine dioxide or chlorine residuals immediately upon sampling. Chlorine dioxide and chlorine

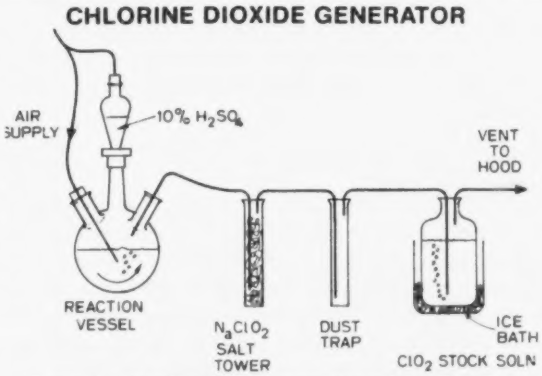

Figure 1. Chlorine dioxide generator using acid activation of a sodium chlorite solution to prepare an approximately 500 mg/l ClO_2 stock solution.

residuals were measured by the iodometric back-titration using an amperometric titrator (Fisher-Porter).

RESULTS AND DISCUSSION

I. Comparison of MPN and MF Methods of Counting Bacteria

This series of experiments had two objectives: 1) a preliminary comparison of the disinfection effectiveness of chlorine dioxide and chlorine according to the kill of natural populations of coliform organisms in wastewater and 2) a comparison of the Most Probable Number Test (MPN) and the Membrane Filter Test (MF) for counting coliform bacteria in wastewater. The disinfectant comparison will be addressed in greater detail in another paper.

Each disinfectant experiment had bacteria counted by both the MPN and MF methods so that both goals were achieved. Duplicate samples were taken for both counting methods. All samples in a given experiment for the MPN test were randomized to eliminate any bias associated with order or time of analysis. The same was done for the MF test.

The comparisons of disinfectants and counting methods were conducted at doses of 5 mg/l chlorine dioxide as mg/l ClO_2 and 5 mg/l Cl_2 as mg/l Cl, and a contact time of 5 minutes. This dose and contact time were chosen to achieve substantial bacterial kills, yet avoid overkill to the extent of no measurable bacterial counts.

The results of these experiments are presented in Table 1. Each "experiment number" indicates a different grab sample of secondary effluent from the Palo Alto Water Pollution Control Plant. The associated values in each cell (Table 1A) are log reduction values of the disinfected sample after a five-minute contact time. The coliform values in Table 1B are logarithms of the replicate bacterial counts in the grab sample before disinfection along with chemical charac-

TABLE 1A: SUMMARY OF LOG REDUCTION Δ (LOG N)

Experiment No.	1 mg/l				Experiment No.	5 mg/l, 5 min contact time							
	ClO_2					ClO_2				Cl_2			
	MPN		MF			MPN		MF		MPN		MF	
	TC	FC	TC	FC		TC	FC	TC	FC	TC	FC	TC	FC
6	0.10	-0.22	-0.17	0.24	8	2.13	1.91	2.79	2.36	2.10	2.93	3.78	3.00
7	0.76	-0.66	-0.29	0.21	9	3.23	3.71	4.39	1.55	2.88	3.10	4.76	2.07
10	0.17	0.07	-0.01	0.16	11	2.86	3.31	2.18	1.94	3.17	3.29	2.63	2.86
16	-0.02	0.07	0.31	0.19	12	1.89	2.34	3.60	2.52	1.86	2.09	3.60	2.28
18	-0.17	0.45	0.15	0.08	14	2.51	2.05	2.48	2.10	2.67	1.94	3.05	2.44
20	0.39	0.08	-0.04	0.14	15	2.52	2.27	2.39	2.10	3.32	3.13	2.78	2.51

Log reduction = log average of number of organisms (N/100 ml) in undisinfected sample minus log average number of organisms in disinfected sample

TABLE 1B: SECONDARY EFFLUENT CHARACTERISTICS

Parameter	Exp. 6	Exp. 7	Exp. 8	Exp. 9	Exp. 10	Exp. 11	Exp. 12	Exp. 14	Exp. 15	Exp. 16	Exp. 18	Exp. 20
Temp., °C	22.6	21.2	22.5	21.2	21.8	21.5	21.8	21.5	22.0	22.8	23.4	22.3
pH	7.15	6.9	7.1	7.3	7.5	7.5	7.45	7.55	7.35	7.8	7.5	7.7
Alkalinity, mg/l as $CaCO_3$	221.	210.	249.	249.	232.	260.	240.	296.0	285.	290.	268.	363.
NH_3-N, mg/l	42.	36.	30.5	42.5	40.	21.6	18.0	19.6	19.5	21.5	22.	24.5
Total COD, mg/l	45.6	34.3	35.9	37.5	41.3	49.0	52.0	46.2	55.3	50.0	39.1	39.0
Total Suspended Solids, mg/l	15.5	14.	8.5	7.5	3.0	24.7	9.5	16.0	7.5	5.5	10.0	21.5
Total Coliforms												
MPN, LOG N/100 ml	5.64	4.81	4.73	4.73	3.70	4.48	3.69	4.97	4.62	4.05	3.95	5.57
MF, LOG N/100 ml	5.93	5.10	4.95	6.27	3.81	3.88	5.51	4.70	4.87	4.69	4.58	5.62
Fecal Coliforms												
MPN, LOG N/100 ml	4.97	3.95	4.47	5.27	3.43	4.23	3.64	4.24	4.43	4.33	4.29	5.24
MF, LOG N/100 ml	4.83	4.92	4.04	3.24	3.49	3.46	4.35	4.26	4.59	4.89	4.65	5.66

teristics of the secondary effluent used for each experiment. The difference between averages of influent and disinfected sample is the reduction, Δ (log N), caused by each disinfectant. The data from these two tables were then subjected to the statistical analysis which follows.

STATISTICAL INTERPRETATION

Two statistical computer packages were used in analysis of the data. The first of these is the Biomedical Computer Programs Package, P-series (BMDP). From this package the BMDP-2V analysis of variance program was used to generate the analysis of variance (ANOVA) tables that follow. The second package used was Minitab II. This package provided the results of the paired t-tests and the correlation tests that also follow.

TABLE 2. CONDENSED RESULTS OF ANOVA TC LOG-KILL DEPENDENT VARIABLE

Source of Variation†	F-Value	Prob. F Exceeded
Experiment	12.37131*	0.008
Method	33.87109*	0.002
Experiment-Method Interaction	15.16024*	0.005

†Disinfectant main effect and disinfectant-experiment interaction omitted
*Significant at 1% level

To examine the differences between the method of counting organisms on log-kill, an ANOVA was run with Total Coliform (TC) log-kill as the dependent variable, and method of counting and experiment (since experiments were performed on different wastewaters and on different days) as grouping variables. This same ANOVA was also run with Fecal Coliform (FC) log-kill as the dependent variable. The results are shown on Tables 2 and 3.

TABLE 3. CONDENSED RESULTS OF ANOVA FC LOG-KILL DEPENDENT VARIABLE

Source of Variation†	F-Value	Prob. F Exceeded
Experiment	2.21334	0.202
Method	7.04393*	0.045
Experiment-Method Interaction	5.2105 *	0.047

†Disinfectant main effect and disinfectant-experiment interaction omitted
*Significant at 5% level

It is apparent from Tables 2 and 3 that there is a significant experiment-method interaction which complicated interpretation of the ANOVA results. This implies that the relationship between log-kill and method of bacterial enumeration is not consistent from experiment to experiment. One suggested hypothesis to explain this inconsistency concerns the effect on the experiments by a variation in Palo Alto wastewater. Correlations of the parameters temperature, pH, alkalinity, ammonia concentration, total COD, and total suspended solids with TC and FC log-kills, however, showed no consistent relationships. It is possible, but unlikely, that an unmeasured parameter or trace industrial constituent may be responsible for the experiment-method interaction.

To exclude the effect of experiment in the comparison of counting methods, a pooled variance paired t-test was run. For this test, a mean of the log number of organisms from each cell was calculated. The differences between method for each experiment were calculated and the mean of these values was subjected to the paired t-test. This comparison was performed over all combinations of parameters. In addition, the log-kill values for fecal and total coliforms were tested. The results, tabulated in Table 4 indicate that, indeed, there is no difference between the MPN and MF Methods of counting organisms in determining the quantity of organisms present.

TABLE 4. COMPARISON OF MPN AND MF METHODS USING A POOLED VARIANCE t-TEST

Analysis	t-Value	Probability t Exceeded	Standard Deviation of Difference
TC Influent	-1.408	0.2182	1.010
FC Influent	1.002	0.3622	0.912
TC Effluent			
ClO_2	-0.322	0.7603	0.393
Cl_2	0.662	0.5373	0.654
FC Effluent			
ClO_2	-0.744	0.4904	0.750
Cl_2	0.541	0.6117	0.641
TC Log-Kill			
ClO_2	-1.231	0.2731	0.892
Cl_2	-1.636	0.1627	1.150
FC Log-Kill			
ClO_2	1.199	0.2843	1.030
Cl_2	0.944	0.3887	0.571

There is, however, an important difference between these two methods. To see this, the standard deviation of each cell is calculated and these standard deviations are weighted and summed over all experiments for both MPN and MF. A comparison of the standard deviation (i.e., precision) between the two methods can be made. The results of such an analysis appear in Table 5. In every case the MPN method shows greater variation than the MF method. This result seems reasonable when one considers that the MPN method itself is based on a statistical probability and not on a direct organism count as is the MF method. The MPN value is the modal value of a 95% confidence interval. Another point to be made from Table 5 is that the magnitude of the difference between two samples as measured by MPN must be greater to be statistically significant than the magnitude as measured by MF [(e.g., (Influent-log) - (Effluent-log) = log-kill)].

TABLE 5. COMPARISON OF METHOD PRECISION

Parameter	Method	$\bar{s}$	$\bar{s}^2$
TC-Influent	MPN	.61	.37
	MF	.11	.01
FC-Influent	MPN	.69	.48
	MF	.14	.02
TC-Effluent ClO_2	MPN	.38	.14
	MF	.23	.05
TC-Effluent Cl_2	MPN	.42	.18
	MF	.21	.04
FC-Effluent ClO_2	MPN	.46	.21
	MF	.13	.02
FC-Effluent Cl_2	MPN	.58	.34
	MF	.13	.02

The mechanisms of disinfection are not completely understood, but most information on the subject indicates that powerful chemical oxidants such as ClO_2 and Cl_2 act destructively upon cell protein, either by inactivation of critical enzyme systems or disruption of protein synthesis directly. Benarde *et al* (1967)(1) have shown that cell wall lysis is not the cause of cell destruction. The disinfection mechanism must then involve at least two steps: 1) chemisorption of the disinfectant at selective active centers on the cell surface, and 2) surface and intrasurface diffusion of the active chemical complex across the cell wall with resultant chemical attack on cellular components. It should be possible, then, for an organism to become "stressed" without being destroyed. If an organism in this stressed state were placed into an environment in which the optimum conditions for growth were present, its chance of survival would be greater than if left in a less favorable environment. This is indeed the case for MPN vs MF methods of organism counting. It has been shown by Mowat (1976) (4) that the MPN method is in fact a more favorable environment for bacterial growth. Therefore, the survival rate of the stressed organisms would be greater in the MPN method than in the MF method, giving results as seen in Table 6. In an actual disinfection situation, the surviving and/or stressed organisms are not subjected to an optimal growth environment, suggesting that the MF method is a more realistic indicator of the number of surviving organisms than is the MPN method.

TABLE 6. COMPARISON OF TWO DISINFECTANTS BY MPN AND MF USING A POOLED VARIANCE t-TEST

Analysis	t-Value†	Probability t Exceeded	Standard Deviation
TC Log-Kill			
MPN	-0.899	0.4099	0.391
MF	-3.514	0.0170*	0.322
FC Log-Kill			
MPN	-0.588	0.5819	0.645
MF	-2.731	0.0412*	0.387

*Significant at the 5% level

†t-Value of $t < 0$ indicates that log kill with $ClO_2 <$ log-kill with Cl_2

SUMMARY

1. The MPN and MF methods of counting total and fecal coliform organisms are equivalent in *accuracy* of measurement.
2. The MF method is a more *precise* method of measurement than the MPN method.
3. The MF method is a more appropriate estimator of realistic situations.

II. Factors Influencing the Bactericidal Efficiency of Chlorine Dioxide

Chlorine dioxide had been observed to have an unexpectedly low efficiency when compared with chlorine in disinfecting Palo Alto secondary effluent (Table 1A). Two experiments were designed to investigate factors that may influence the performance of chlorine dioxide as a disinfectant: 1) suspended solids and disolved constituents present in the wastewater as well as 2) the nature of the indicator organism.

EFFECT OF SUSPENDED SOLIDS

Experiment "A" was designed to study the effect of suspended solids on disinfection. Two parallel reactors were dosed with 5 mg/l chlorine dioxide. One reactor contained secondary effluent with 12 mg/l suspended solids. The second reactor contained no measurable suspended solids. The reactor containing the suspended solids showed less bacterial kills than the reactor containing settled secondary effluent (Table 7). These results indicate that the suspended solids offered some physical protection to the bacteria, and possibly exerted an additional chlorine dioxide demand. The difference between these two systems, however, is not dramatic.

TABLE 7. EXPERIMENT A: EFFECT OF SUSPENDED SOLIDS ON DISINFECTION WITH 5 mg/l ClO_2, 5-MIN CONTACT TIME

		MPN		MF	
		TC	FC	TC	FC
Secondary Effluent	N_{INF}	4.45	4.45	4.65	4.88
	N_{EFF}	1.30	1.30	2.21	2.30
	ΔN	3.15	3.15	2.44	2.58
Settled Secondary Effluent	N_{INF}	4.88	4.88	4.67	4.79
	N_{EFF}	1.54	1.45	1.28	1.09
	ΔN	3.34	3.43	3.39	3.70

	Chemical Characteristics					
	Temp. °C	pH	Alk. mg/l	NH_3-N mg/l	COD_T mg/l	TSS mg/l
Secondary Effluent	22.3	7.3	290.4	18.5	46.4	12.0
Settled Secondary Effluent	22.3	7.4	290.4	18.5	33.0	ND*

*Not detectable

EFFECTIVENESS AS A BACTERICIDE

Experiment "B" was designed with two purposes in mind: 1) to provide a link with the literature that reports superior bacterial kills with ClO_2, and 2) to indicate whether the dissolved constituents in the Palo Alto secondary effluent exert a demand for ClO_2. Sterile buffered deionized water (D.I.) and sterile filtered (0.45μ) secondary effluent were inoculated with 10^8 fecal coliforms/ml. The bacteria had been obtained from the completed MPN procedure and purified to yield a strain of *Escherichia coli*. This culture was then grown overnight in nutrient broth, centrifuged, and resuspended in sterile buffer water.

The data are summarized in Table 8, along with similar data from the work of Walters (1976)(6). A comparison of the secondary effluent results with Walters' data indicates that our method of bench-scale disinfection is comparable with reported studies that use a "clean system" with a pure inoculum. Also, these two systems are identical with the possible exception of the type of pure inoculum used: a "fecal strain of coliform" which we used versus a culture of *Escherichia coli B* used by Walters. While one experiment is inconclusive, the variable resistance of strains of one bacterial type seems to be important. Hence, the validity of predicting the disinfection performance on a mixed, native population of bacteria in wastewater based on results of pure culture studies is questioned.

TABLE 8. EXPERIMENT B: EFFECT OF BACTERIAL CULTURE AND DISSOLVED SUBSTANCES ON DISINFECTION WITH 5 mg/l ClO_2.

Contact Time	Log Number Bacteria/100 ml		
	Data of Walters (1976)(6)	D.I. H_2O	Secondary Effluent
0	7.91	8.49	8.54
5 min	3.78	1	1.43
10 min	3.58	0	0.3

	Chemical Characteristics			
Sample	pH	Alk. mg/l	NH_3-N mg/l	COD_T mg/l
D.I. H O	6.5	10.3	0	8
Second. Effl.	8.2	289.2	45	39

A comparison also can be made between the D.I. water and secondary effluent. After a 5-min. contact time, which should ensure complete mixing, there is no substantial difference in bacterial kills. Therefore, in this sample of wastewater, there seems not to have been any inhibition of disinfection caused by dissolved substances in the secondary effluent.

Based on the results of these experiments, a study was designed to verify the comparison between a pure culture and a native population, to compare two wastewaters and their respective coliform populations, and to compare chlorine and chlorine dioxide on pure cultures of *Escherichia coli*.

The first comparison, a pure culture of *E. coli* with a native coliform population, verified the results obtained earlier. Namely, pure batch-grown cultures of *E. coli* seem to be more susceptible to disinfection than the native populations from which they were isolated. Samples of secondary effluent, settled and containing non-settling floc particles, were blended in a Waring blender for 30 seconds to minimize any physical protection afforded by aggregation (Moffa and Smith, 1975)(3). These samples were then dosed with chlorine dioxide and chlorine, as were samples of filter-sterilized secondary effluent and buffered D.I. water that contained an inoculum of *E. coli*. The results are summarized in Table 9 and are similar to the results obtained earlier and indicated in Table 8. Again, pure batch-grown cultures of *E. coli* seem to be more susceptible to disinfectants than the native populations from which they were isolated. Also, the data indicate that there is not a significant demand exerted by dissolved substances in the effluent relative to demand exerted by the disinfection process, since both the filter-sterilized effluent and buffer water yield approximately the same log survival. This had also been observed earlier and is noted in Table 9. Finally, a comparison of the influent ("0-time") fecal coliform counts between the blended secondary and settled, blended secondary effluents reveals that most coliform organisms are not associated with settleable floc-type particles. Instead, these bacteria seem to be present as more dispersed growth, or associated with smaller suspended particles. Therefore, these indicator organisms are generally less protected than other bacteria which are enriched selectively in the activated-sludge process by virtue of their ability to flocculate.

Another comparison between two different wastewaters was made to verify the data relating to the relative inefficiency of chlorine dioxide as a bactericide in secondary effluent. Effluent from San Jose Wastewater Treatment plant, a 3.4 x $10^5 m^3/d$ (90 mgd) activated sludge plant, was compared with Palo Alto secondary effluent which has been used throughout this feasibility study. The data are summarized in Table 10. The coliform survival data (Table 10A) are similar to the results that have been obtained with Palo Alto secondary effluent.

The performance of chlorine dioxide and chlorine are nearly identical when compared using pure cultures in secondary effluent (Table 10B). This result has been observed before (Table 9), but is contrary to other information (Bernarde et al., 1967; Walters, 1976) indicating that chlorine dioxide is a more efficient bactericide than chlorine. There was a greater demand for chlorine dioxide in San Jose effluent than in Palo Alto effluent. However, this seemed to have a minimal effect on the bactericidal effectiveness. The equal disinfection ability of chlorine dioxide and chlorine is still unexplainable; the only apparent difference between the compared experiments is the type of organism and possibly,

TABLE 9. EFFECT OF BACTERIAL CULTURE ON DISINFECTION WITH 5 mg/l ClO_2 AND 5 mg/l Cl_2

Contact Time	LOG SURVIVAL							
	Native Population of Fecal Coliforms				Pure Cultures of *E. coli*			
	Blended Secondary Effluent		Settled, Blended Secondary Effluent		Filter-Sterilized Secondary Effluent		Sterile, Buffered D.I. Water	
	ClO_2	Cl_2	ClO_2	Cl_2	ClO_2	Cl_2	ClO_2	Cl_2
0	4.6	4.6	4.5	4.8	8.3	8.4	8.0	8.6
2	1.9	3.8	2.0	2.5	6.4	7.2	6.8	6.7
5	1.3	2.4	0	2.0	1.0	—	0	0
10	1.0	2.1	0.3	1.3	0	1.2	0.5	0

TABLE 10. PALO ALTO vs SAN JOSE SECONDARY EFFLUENT 5 mg/l ClO_2 AND Cl_2

A. Secondary Effluent, Native Populations

Contact Time, minutes	SAN JOSE			
	ClO_2		Cl_2	
	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l
0	4.8		4.8	
2	2.2	1.64	1.9	3.37
5	2.0	1.70	1.7	3.31
10	1.8	1.58	1.6	3.32

TABLE 10. PALO ALTO VS SAN JOSE SECONDARY EFFLUENT 5 mg/l ClO_2 AND Cl_2 continued

B. Filter-Sterilized Secondary Effluent, Pure Cultures of *E. coli*

	SAN JOSE				PALO ALTO			
	ClO_2		Cl_2		ClO_2		Cl_2	
Contact Time, minutes	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l
0	8.5	—	8.4	—	8.5	—	8.6	—
2	1.9	1.64	1.4	3.34	0	3.48	2.8	3.71
5	1.1	1.35	0	3.45	0	2.76	0	3.79
10	0	1.56	0	3.21	0	2.33	0	3.74

C. Sterile Buffered Water, Pure Cultures of *E. coli*

	SAN JOSE				PALO ALTO			
	ClO_2		Cl_2		ClO_2		Cl_2	
Contact Time, minutes	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l	Fecal Coliforms N/100 ml	Residual mg/l
0	8.2	—	8.1	—	8.6	—	8.6	—
2	0	4.81	0	4.23	3.5	4.64	2.8	4.29
5	0	4.60	0	4.12	0	4.32	0	4.09
10	0	4.32	0	4.06	0	4.13	0	3.94

Note: All bacterial counts are log number surviving at the given contact time; residual Cl_2 is as total available Cl_2, residual ClO_2 is as ClO_2.

the conditions used to culture the populations. These variables and mechanisms of disinfection will have to be studied further to explain the discrepancies.

Finally, two different populations of *E. coli*, isolated from San Jose and Palo Alto secondary effluent, were compared. No significant difference was observed in log-kills of the two different cultures. Therefore, it seems that the method of culturing pure populations of bacteria used in disinfection studies as opposed to the source of the population may be important regarding mechanisms of disinfection.

SUMMARY

1. Suspended solids offer some physical protection to the bacteria from the action of chlorine dioxide.
2. Pure batch-grown cultures of *E. coli* are more susceptible to ClO_2 and Cl_2 than the native populations from which they were isolated, and the method of culturing pure populations of bacteria may be important regarding responses to disinfectants.
3. Chlorine dioxide and chlorine are nearly identical when comparing kills of pure cultures of *E. coli* or native populations of fecal coliforms in either Palo Alto or San Jose secondary effluent.

III. Virucidal Effectivensss of Chlorine Dioxide

Selected experiments throughout the study were conducted to assess the virucidal effectiveness of chlorine dioxide as compared with chlorine. Coliphage and Poliovirus I were inoculated into secondary effluent, then dosed with 5 mg/l chlorine dioxide and chlorine. Samples were taken at 2-, 5-, and 10-minute contact times and analyzed for virus and fecal coliforms by Dr. Robert Cooper, University of California at Berkeley. The results are shown in Figure 2. In both trials, chlorine dioxide was a much more effective virucide than was chlorine, although the coliform survivals would indicate that both disinfectants were performing equally well.

The log reductions of the coliphage and poliovirus by each of the disinfectants was similar, a response that had been observed in preliminary experiments. Both phage and virus experienced significant log reductions by chlorine dioxide, and both were affected less by the same dose of chlorine. Since the test procedure for poliovirus is much more expensive and time consuming than the recovery procedure for coliphage, the coliphage was chosen as an indicator of virus response to both disinfectants. The coliphage had been isolated from this secondary effluent, as was described earlier. Therefore, *in situ* phage, which may be more realistic indicators of native populations of other pathogens, was used as opposed to inocula of laboratory-grown strains of organisms.

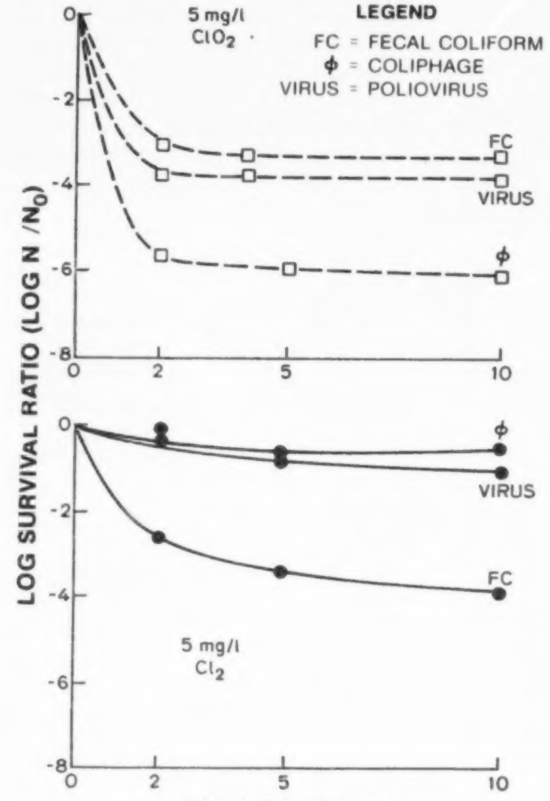

Figure 2. Comparison of *In situ* coliphage and an inoculum of Poliovirus I in secondary effluent.

The remaining virus experiments were conducted in parallel with a large grid of experiments designed for modeling dose-contact time relationships for chlorine dioxide and chlorine. A full factorial experiment in randomized block design was selected. Three dosage levels (2, 5, and 10 mg/l) of each disinfectant, and three contact times (5-, 15-, and 30-minutes) were tested in eight replicate experiments. Reductions of total coliform organisms were measured in all experiments, and coliphage reductions were measured as well in Experiments 1 and 8. The order of choice of disinfectant and dose and the sequence of analyses for bacterial and viral counts were all randomized.

The reactor for these experiments was a 4-liter aspirator bottle which had been modified with 4 identations

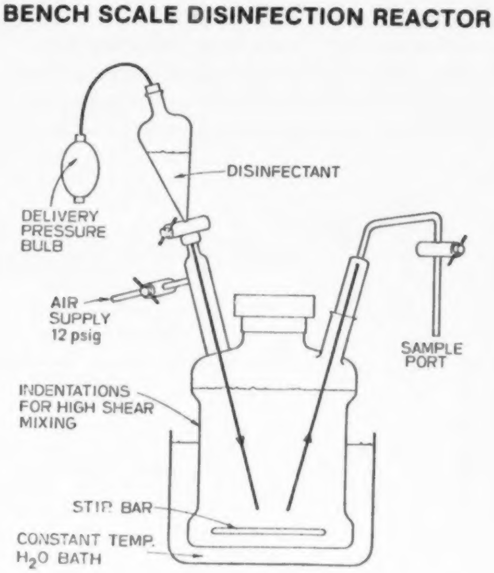

Figure 3. A rapid-mix, rapid-sampling, 4 liter reactor. Dye studies indicate that complete mixing takes place in less than 5 sec.

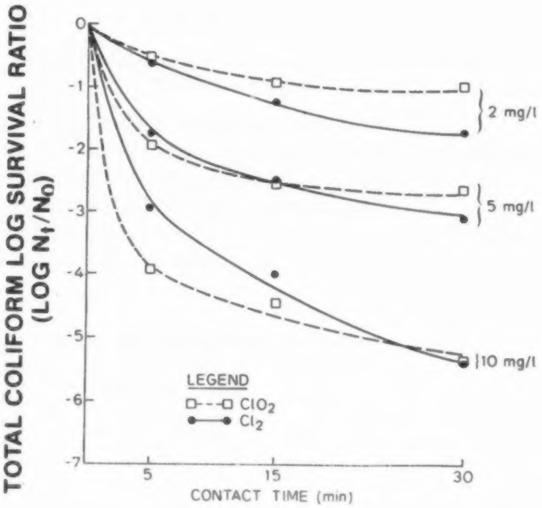

Figure 4. Total coliform survival comparing ClO_2 at 3 doses and 3 contact times in secondary effluent.

similar to those of a trypsinizing flask (Fig. 3). The reactor was also equipped with an injector and sampler that were located in the most active mixing zone of the reactor. All connections were glass or teflon. The vessel was pressurized (12 psig) and both disinfectant and sample delivery were pressure driven. A dye study indicated that complete mixing of injected disinfectant liquid and the wastewater occured in less than five seconds. The reactor was placed in a constant-temperature bath and all experiments were conducted at 24±0.5°C.

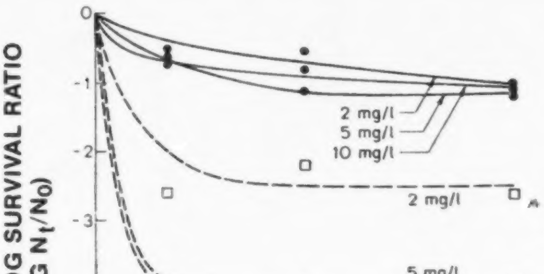

Figure 5. ***In situ* coliphage survival in secondary effluent at 3 doses and 3 contact times.**

The results of the virus analyses that were run parallel with the total coliform analysis are shown in Figures 4 and 5. Wastewater characteristics are summarized in Table 11. Once again, the total coliform bacteria respond similarly to the stress of a given disinfectant dose (Fig. 4), whereas all three doses of chlorine dioxide (2, 5, and 10 mg/l) are much more effective at inactivating the coliphage than even the highest dose (10 mg/l) of chlorine (Fig. 5). This suggests that accepted indicator organisms (total coliform bacteria) may yield conservative, possibly inaccurate, performance data for disinfection with chlorine dioxide in secondary effluent

TABLE 11. WASTEWATER CHARACTERISTICS

Parameter	Experiment Number	
	1	8
Temperature, °C	24	24
Total Filterable Residue, mg/l	28.0	23.5
Chemical Oxygen Demand, mg/l	30.8	34.0
Alkalinity, Total mg/l as Ca CO_3	222.1	247.9
pH	7.4	7.0
Nitrogen: Ammonia-N, mg/l	29	34

A final set of experiments was run to assess the biocidal effectiveness of both disinfectants at very short contact times, since the shortest contact time studied had been 2 minutes (Fig. 2). These experiments were run at 5 mg/l doses ClO_2 and Cl_2 and sampled at 15, 45, 90, and 120 seconds. The results (Fig. 6) again indicate that ClO_2 is a more effective virucide than a bactericide, and that ClO_2 performs better than Cl_2 in a short contact time (<120 seconds) in secondary effluent.

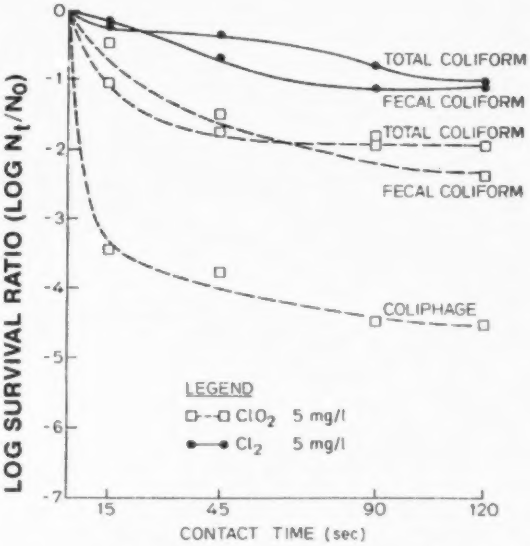

Figure 6. A comparison of the responses of several organisms to ClO_2 and Cl_2 at very short contact times in secondary effluent.

The differential behavior of chlorine dioxide compared with chlorine and of bacteria compared with virus in response to chlorine dioxide in secondary effluent merit further investigation. A study of the factors that influence mechanisms of inactivation by ClO_2 in a complex matrix such as secondary effluent may have important implications regarding the design of processes for disinfection.

SUMMARY

1. *In situ* coliphage and inocula of Poliovirus I exhibited the same response to both chlorine dioxide and chlorine dosed at 5 mg/l in secondary effluent.
2. Coliphage inactivation at all doses of chlorine dioxide (2, 5, and 10 mg/l) is much greater than the highest dose of chlorine (10 mg/l) in secondary effluent.
3. Chlorine dioxide is a more effective virucide than a bactericide, and chlorine dioxide performs equal to or better than chlorine at all contact times (15 sec. to 30 min.) and doses (2 to 10 mg/l) tested.

CONCLUSIONS

1. The MPN and MF methods of counting TC and FC are equal in accuracy of measurement; however, the MF method is more precise.
2. The variable resistance to disinfection of types of indicator organisms and types of populations chosen for a disinfection study is critical.
3. Chlorine dioxide is a significantly more effective virucide than is chlorine in secondary effluent, although disinfection efficiency as measured by coliform survival indicates that ClO_2 is not different from Cl_2.

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10.

CHLORINE DIOXIDE: ANALYTICAL MEASUREMENT AND PILOT PLANT EVALUATION

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INTRODUCTION

Chlorine has long been favored as a disinfectant in both water and wastewater treatment by virtue of its bactericidal effectiveness, low cost and convenience, and relatively long-lived residual. However, chlorination as normally practiced in water and wastewater treatment has been found to result in the formation of trihalomethanes and other chlorinated organics that are undesirable from the viewpoint of water pollution control in general and human health specifically (11).

The EPA is actively seeking substitutes for chlorine as a disinfectant. Chlorine dioxide has received considerable attention in this respect. Chlorine dioxide has been used most extensively in this country by the pulp and paper industry as a bleaching agent and to a small extent for the control of taste and odor problems in a few water treatment systems (14). Chlorine dioxide has been found in a number of investigations to be equal or superior to chlorine as a disinfectant (1, 2, 9, 10, 13). However, most of the studies were conducted in clean systems or synthetic sewages inoculated with lab-grown strains of bacteria. These results may not be directly applicable to typical municipal wastewaters.

The topics from this research that will be presented here include:

1. Analytical methods for the measurement of both stock and residual concentrations of chlorine and chlorine dioxide.
2. The design and results of a dose-time matrix experiment in which chlorine and chlorine dioxide are compared in the disinfection of a municipal secondary effluent.
3. The development from the results of the dose-time matrix experiment of a simple disinfection model for chlorine or chlorine dioxide.
4. Plans for upcoming verification of the laboratory-derived model on a pilot plant scale.

ANALYTICAL DETERMINATION OF CHLORINE AND CHLORINE DIOXIDE

Measurement of chlorine dioxide in the mg/l range is a relatively new field. The experience gained over many years in the study of chlorine and its residual forms provides a starting point for the development of a chlorine dioxide residual measurement technique. The following is a brief description of the stock generation techniques, stock measurement techniques, and residual measurement techniques for both chlorine and chlorine dioxide as used in this study.

Chlorine stock solutions were generated by bubbling chlorine gas through chilled distilled water. Chlorine dioxide solutions were generated by the sulfuric acid activation of sodium chlorite as described in *Standard Methods* (12). In order to insure a pure chlorine dioxide solution, the generated chlorine dioxide gas was passed through a tower of sodium chlorite flakes and collected in chilled distilled water, thereby removing any chlorine impurity. Both chlorine and chlorine dioxide stock solutions were standardized (in triplicate) before each experiment using the iodometric technique prescribed in *Standard Methods*. The concentration of chlorine dioxide stock varied from 300 mg/l to 600 mg/l. The concentration of chlorine stock varied from 3000 mg/l to 8000 mg/l.

Several methods are available for the measurement of chlorine residuals in wastewater. These methods have been extensively developed over many years and the strengths and weaknesses of each are reasonably understood. Many of these same methods are also applicable to the measurement of chlorine dioxide residuals, with some modifications. Two methods for residual measurement of chlorine dioxide were chosen for investigation in this study: 1. DPD—Ferrous Titrimetric Method, and 2. Amperometric Method.

DPD—Ferrous Titrimetric Determination of Chlorine Dioxide Residuals

In attempting to measure residual chlorine dioxide by the DPD-FAS technique (6, 7, 8), some difficulties were encountered. The residual as measured by DPD was consistently higher than the chlorine dioxide dose. The stock solution was standardized immediately before use by the iodometric method, and the dose calculated on this basis. Not only was this discrepancy seen in wastewater, but also in deionized water, distilled water and tap water. This discrepancy was not seen when chlorine was the oxidant of interest.

In the iodometric method, chlorine dioxide is reduced to chloride (Cl^-). However, several researchers (6, 9, 10) have reported that the pH used in the iodometric analysis ($pH \approx 1.8$) is not low enough to effect complete reduction of the chlorine dioxide to Cl^- (see Fig. 1). If indeed, this was the case, the iodometric results would give a low value for the concentration of chlorine dioxide in the stock solution. The DPD method, which is performed at pH of 6.2-6.4 is assumed to measure the reduction of chlorine dioxide to chlorite, and would therefore indicate a higher value of chlorine dioxide than that given by the iodometric analysis.

$$ClO_2 + e^- \longrightarrow ClO_2^- \qquad pH\ 7$$

$$ClO_2 + 5e^- \longrightarrow Cl^- + 2\,O^{--} \qquad pH < 2$$

FIGURE 1. Reduction of ClO_2 According to pH.

To test this hypothesis, two experiments were performed. In the first experiment, the iodometric analysis was performed at various pH's ranging from 2.0 to 0.8. No difference in titration value was found. In the second experiment, the iodometric analysis performed at pH 1.8 was compared with the amperometric analysis at pH 7. The amperometric method is analogous to the iodometric method with these exceptions:

1. In the amperometric analysis the end-point is detected by observing the change in current flow in the sample while adding a strong reducing agent as titrant.
2. In the amperometric analysis phenylarsine oxide is the titrant instead of sodium thiosulfate as used in the iodometric method.
3. In the amperometric analysis the determination is performed at pH 7. At this pH only one electron transfer is possible for chlorine dioxide.

Since at a pH of 7, chlorine dioxide is only reduced to chlorite, a higher concentration of chlorine dioxide as measured by the amperometric analysis would indicate that chlorine dioxide was not being fully reduced to chloride as assumed in the iodometric method. The analyses were run in distilled water. The results appear in Table 1, and a statistical analysis appears in Table 2.

TABLE 1. COMPARISON OF AMPEROMETRIC AND IODOMETRIC METHODS FOR DETERMINING CONCENTRATION OF ClO_2

Trial	Iodometric*	Amperometric*
1	563.7	547.2
2	563.7	554.8
3	566.4	571.9
4	561.0	560.5
5	560.3	558.6
Mean	563.0	558.6
S.D.	2.18	8.06
S^2	4.77	64.98

*mg/l AS ClO_2.

TABLE 2. COMPARISON OF AMPEROMETRIC AND IODOMETRIC METHODS BY ANALYSIS OF VARIANCE

Source of Variance	Degrees of Freedom	Sum of Squares	Mean Square	F-Ratio
Between Methods	1	48.8	48.8	1.12
Error	8	348.7	43.6	
TOTAL	9	397.5		

As can be seen from Table 2, the difference between the values obtained by the two methods is not significant. It can be inferred from these data that the iodometric method accurately measures the concentration of chlorine dioxide in stock solutions. The inconsistency must then be in the assumed reactions of the DPD-FAS method. It is hypothesized (although untested at this time) that the chlorite from the reduction of chlorine dioxide is reacting with the DPD reagent to a slight extent. Due to the continuing problems with the DPD-FAS method, further investigations into utilizing the amperometric method for measurement of residual chlorine dioxide were made.

Amperometric Determination of Chlorine Dioxide Residuals

To investigate the possible interference of chlorite in the determination of chlorine dioxide by the amperometric method, a solution of sodium chlorite was added to a sample containing chlorine dioxide in distilled water. Since the sodium chlorite used was only ~ 95 percent pure, glycine (aminoacetic acid) was added to the sample to react any chlorine or bromine compounds present which would give an apparent positive result from the chlorite. Results of this experiment are reported in Table 3.

As seen in Table 3, chlorite does not interfere with the chlorine dioxide measurement.

TABLE 3. THE EFFECT OF CHLORITE ON THE DETERMINATION OF CHLORINE DIOXIDE BY AMPEROMETRIC TITRATION

Sample	ClO_2	ClO_2 + 4.33 mg/l ClO_2^- + Glycine*
1	5.47	5.55
2	5.55	5.51
3	5.72	5.55
4	5.61	5.61
5	5.59	5.61
Mean	5.59	5.57

* Two ml of 10% W/V solution of Glycine added to 200 ml of sample.

The amperometric titration measures iodine (I_2) released into solution by the oxidation of iodide by any powerful oxidant such as chlorine dioxide or chlorine. If this released I_2 comes into contact with organic constituents in the sample (as in the case of wastewater), the I_2 will react with the organics. To prevent this, a back-titration procedure is recommended, in which the liberated I_2 is immediately reacted with phenylarsine oxide (PAO), which has been added to the sample prior to the addition of iodide (as KI). The PAO is added in excess and the unreacted PAO is back-titrated with an I_2 solution of known concentration. The concentration of chlorine dioxide (or chlorine) is then determined by difference. To evaluate the need for this back-titration procedure, chlorine dioxide residual measurements were made in Palo Alto secondary effluent.

The results of this experiment are reported in Table 4. The experiment involved dosing 2 liters of Palo Alto secondary effluent to 4.50 mg/l with pure chlorine dioxide. The sample was then contacted for 2 minutes. After the contact period, sufficient KI and pH 7 buffer were added to the sample to react with all the chlorine dioxide present. In the case of the I_2 back-titration, sufficient PAO was added to swamp the released I_2. Eight (8) replicates were run at approximately one-minute intervals. The experiment was repeated for filtered Palo Alto secondary effluent. The values reported have been corrected for blank titrations. In the case of the forward titration, no correction was necessary, but the I_2 back-titration more I_2 solution was required in the blank than was indicated by

TABLE 4. AMPEROMETRIC TITRATION EVALUATION*

	Secondary Filtered		Secondary Unfiltered	
Sample	Forward Titration	Back Titration	Forward Titration	Back Titration
1	1.90	2.12	1.62	1.88
2	1.86	2.31	1.62	1.97
3	1.90	2.13	1.52	2.07
4	1.84	2.22	1.52	1.88
5	1.81	2.22	1.52	1.88
6	1.73	2.12	1.48	1.88
7	1.67	2.12	1.39	1.88
8	1.91	2.12	1.27	1.79
Mean	1.80	2.17	1.49	1.90
S.D.	±0.083	±0.067	±0.109	±0.077
S^2	0.0069	0.0045	0.0119	0.0060

* Results are given as mg/l ClO_2 at 2-minute contact time; dose = 4.50 mg/l ClO_2.

TABLE 5. ANALYSES OF VARIANCE FOR AMPEROMETRIC TITRATION

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F-Ratio
ANOVA for filtered secondary effluent:				
Between forward and back titrations	1	.54023	.54023	82.79*
Error	14	.09135	.006525	
TOTAL	15	.63158		
ANOVA for unfiltered secondary effluent:				
Between forward and back titrations	1	.67651	.67651	66.35*
Error	14	.14274	.01020	
TOTAL	15	.81924		

*$P < .001$ that F is exceeded.

the amount of PAO added. (The I_2 solution was standardized immediately prior to use). Although not shown in Table 4, the blank titrations (done in triplicate) showed the same variation as the forward or backward titration. The standard deviations shown in Table 4 are not statistically different from one another as measured by the F-test.

Table 5 gives two analyses of variance tables comparing the forward titration with the I_2 back-titration. As indicated, there is a significant difference between the values measured by the two titrations. Based on these results, the amperometric back-titration method was chosen for the measurement of chlorine dioxide residuals in wastewater.

Amperometric Determination of Chlorine Residuals

The amperometric back-titration method for measurement of chlorine residuals in wastewater is a widely used and well-accepted method. Since this method was found acceptable for chlorine dioxide residual measurements, it was also chosen for the measurement of chlorine residuals. The analyses for chlorine residuals (as Total Available Chlorine) are performed at pH 4 to insure quantitative response from all forms of combined chlorine. At this pH there is no need for a blank correction. The reaction of PAO with I_2 must be somewhat slower at pH 7 than at pH 4, so that some of the added I_2 titrant would be available for reaction with other sample constituents; hence the need for blank correction in determining chlorine dioxide residuals at pH 7.

Future Requirements

The amperometric back-titration procedure reported here has proved adequate for the needs of this study. There is, however, a continuing need for a residual measurement technique capable of differentiating between chlorine dioxide and chlorite in the same wastewater sample. Chlorite is suspected of being a human health hazard, specifically of being a cause of methemoglobinemia. In addition, the use of chlorine dioxide in conjunction with chlorine may be the most economically attractive means of pathogen destruction in wastewater, so that an analytical method to differentiate chlorine dioxide from chlorine would be required to control the disinfection process. An ideal method, then, would be able to differentiate chlorine (as chloramines), chlorine dioxide, and chlorite in a wastewater matrix. Work is continuing in this area with the amperometric technique as the primary analytical tool.

COMPARATIVE BACTERICIDAL EFFECTIVENESS OF CHLORINE AND CHLORINE DIOXIDE

Reactor Design

To compare the relative effectiveness of chlorine dioxide with that of chlorine, secondary effluent from the Palo Alto Water Pollution Control Plant was dosed with various concentrations of either chlorine or chlorine dioxide. The reactor for these experiments was a specially modified four-liter aspirator bottle. The modifications include four indentations similar to those of a trypsinizing flask, oxidant injector and sample ports located in the most active mixing zone of the reactor, and all glass or teflon connections. The reactor was maintained at 15 psig and both disinfectant injection and sample delivery were pressure driven. Stirring was provided by a 2½" magnetic stir bar (cylindrical) with ratational speed of ~ 600 rpm. The reactor was maintained at 24°C for all experiments.

To evaluate the dispersion efficiency of added chemicals within this batch reactor, a dye-train study was performed. The dye-train study involved dosing three liters of deionized water with a methylene blue stock solution of 500 mg/l. Sufficient stock was added to make a final concentration of 5 mg/l in the reactor. These conditions mimicked those in which actual disinfection experiments were performed. Samples were taken at 2, 5, 10, 20, 40, 60 and 120 seconds and analyzed spectrophotometrically for methylene blue. The results of this experiment indicate that complete mixing is effected in less than five seconds.

Experimental Design

The experiments for the comparison of bactericidal effectiveness of chlorine and chlorine dioxide were performed on wastewaters from the Palo Alto Water Pollution Control Plant. Secondary effluent was collected at about 8:00 AM on the day of each experiment. Eight separate experiments were performed on five different days over a two-week period. On three days, two experiments were run on the same wastewater. On two days, only one experiment was run. An experiment is a complete dose-time matrix in which either chlorine or chlorine dioxide is added to wastewater at a dosed concentration of 2, 5 or 10 mg/l. All of the six runs within an experiment (two disinfectants, three doses) were done in randomized order. Samples were taken at 5-, 15- and 30-minute intervals for both bacterial analysis and residual disinfectant determination.

Samples for bacteria analysis were also taken at time 0. Table 6 graphically illustrates the dose-time matrix of this study.

TABLE 6. EXPERIMENTAL DOSE-TIME MATRIX FOR CHLORINE CHLORINE DIOXIDE COMPARISON

		CHLORINE DOSE mg/l			CHLORINE DIOXIDE DOSE mg/l		
		2	5	10	2	5	10
CONTACT TIME	5 MIN						
	15 MIN						
	30 MIN						

Experimental Results

All samples for bacterial analysis were analyzed by the membrane filter technique for total coliform bacteria (12). Four dilutions of each sample were analyzed in order to obtain an incubated sample plate with a significant number of colonies to count but a sufficiently small number, so that the colonies do not overlap (usually 20-80 colonies are adequate). It is possible, then, to have from one to four measurements of coliform bacteria for each sample. The logarithms of the numbers of bacteria for each sample are given in Table 7 for samples disinfected with chlorine and in Table 8 for samples disinfected with chlorine dioxide. Where more than one value appears for each sample, the individual values represent bacterial counts from different dilutions but from the same sample.

Statistical Analysis of Experimental Results

Preliminary statistical analyses were done on the data from Tables 7 and 8. To investigate any biases in the data due to the order of experiments or sampling technique, the data from time 0 were subjected to an analysis of variance (ANOVA). The grouping variables used were wastewater, disinfectant, and disinfectant dose. In setting up the ANOVA in this way, all experiments are considered but there is no difference between experiments except the wastewater on which the experiment was run. The results of this ANOVA are shown in Table 9.

The only effect that is significant is that of wastewater, indicating that the variance in bacterial numbers at time 0 for each experiment is due only to the difference inherent in the wastewater used for that particular experiment. No significant correlation was found between measured wastewater parameters (Total Filtrable Residue, COD, Alkalinity, pH, ammonia-nitrogen) and bacterial counts.

The next step in the statistical analysis was to investigate any differences between experiments performed on the same wastewater; that is, the reproducibility of experiments. Again, analysis of variance was used with grouping variables of wastewater, experiment, disinfectant, disinfectant dose, and contact time. The ANOVA is reported in Table 10. As seen in this ANOVA, all variables *except* experiment have a significant

TABLE 7. LOG [SURVIVING BACTERIA] IN EIGHT REPLICATE EXPERIMENTS USING CHLORINE AS DISINFECTANT

Dose:	2 mg/l								5 mg/l								10 mg/l							
Experiment:	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
0 MIN	5.85	6.09	6.00	6.43	6.81	6.63	6.62	5.89	5.78	6.14	6.10	6.79	6.83	6.67	6.63	5.83	5.92	6.30	6.08	6.70	6.84	6.67	6.60	5.73
	5.90	6.28	6.18	6.74				5.90	5.66	6.32	6.36					6.30	6.00		5.84					5.75
								5.78	5.30							5.60								5.30
								5.48																
5 MIN	4.78	5.75	5.95	6.16	6.13	6.30	6.34	5.37	4.09	3.83	3.96	4.15	4.15	4.53	3.90	3.86	2.91	2.51	2.34	3.77	2.81	3.24	3.24	3.00
	5.00	5.30	5.70	6.48	6.15	6.28		5.40	4.08	3.90	4.08	4.06	4.15		4.26	4.06	2.81	2.26	2.40		2.69	3.08		2.67
								5.60	3.60	4.08	3.78	4.41	4.30			4.08		2.16	2.42		2.42			
										3.95	3.70	4.14	4.49			3.95		2.07	2.09					
15 MIN	4.48	4.70	4.92	5.34	4.72	5.45	5.41	4.60	2.84	2.28	2.36	2.78	3.49	4.26	3.85	3.01	2.27	1.00	1.74	1.34	2.04	2.24	2.22	2.04
		4.62	4.79	5.25	4.46	5.62	5.55	4.58	3.15	2.00	3.23	3.34	3.70	4.79	3.75	3.40	1.81	1.41	1.76	1.30		1.91	1.79	1.66
		4.30	4.90	5.72	4.78	5.68			3.48	3.30		3.48	3.30			3.34	1.76	1.20	1.50			1.75	1.91	1.41
		4.30	4.84	5.53	4.48							4.30				3.60	1.57	1.04	1.48					
30 MIN	4.34	3.30	3.30	4.70	4.00	4.78	4.41	4.00	2.71	1.88	1.76	2.91	3.23	2.86	3.30	2.34	1.04	0.60	1.15	1.53	0.30	0.95	1.23	0.48
			3.30	4.58	3.70	4.70	4.80	3.95	2.48	1.90	1.30	2.87	3.21	2.89	3.11	2.73	0.60	0.60	0.34	1.30		0.70	1.26	0.34
			4.00	4.60		5.00	4.78	4.00		1.30	2.15	3.38	3.20	2.84	3.00	2.86	0.48	0.54						0.48
				4.00		4.85	5.08				2.00					2.60								

CHLORINE DIOXIDE

TABLE 8. LOG [SURVIVING BACTERIA] IN EIGHT REPLICATE EXPERIMENTS USING CHLORINE DIOXIDE AS DISINFECTANT

	DOSE :	2 mg/l								5 mg/l								10 mg/l							
	EXPERIMENT :	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
CONTACT TIME	0 MIN	5.91 5.90	6.05 6.18	6.09 6.45	6.73	6.79	6.41 6.67	6.48	5.85 5.91 5.90 5.70	5.83 5.60 5.00	6.02 6.28	6.26	6.83	6.76	6.52	6.67	5.77 5.88 5.00	5.87 5.60	6.12 6.38	6.15 6.20	6.83	6.70	6.73	6.93	5.93 5.75
	5 MIN	5.08 5.95	4.41 4.36 4.30 5.00	4.34 4.50	5.00 4.97 5.08 5.00	4.99 4.99 5.11 5.00	5.21 5.02 5.34 5.00	5.08 4.94 5.41 5.00	5.03 5.04 5.30	3.99 3.91	2.00	2.78 3.30	3.76 3.90 3.60 3.30	3.45 3.75 4.15 4.04	3.41 3.30 3.00	3.98 4.08 3.30 3.30	3.20 3.30 3.70	1.90 1.95 2.18 2.04	1.30 1.00 0.30	1.78 1.78 1.56	2.62 2.46	2.15 2.15 1.70	2.26 2.56 2.32 1.98	2.45 2.43 1.95	1.60 2.08 1.65 1.53
	15 MIN	5.26 5.00	4.38 4.38 4.48	4.38 4.26 4.30	5.06 4.95 4.78 5.04	5.25 5.44 5.15 5.00	5.17 5.00 5.48 5.15	5.20 5.09 5.38 5.26	4.93 4.61 4.60 4.78	3.23 3.20 3.00	2.00	1.70	2.38 2.27 2.30 3.00	2.89 3.15 2.90	3.03 3.82 3.72 3.70	2.28 2.28 3.00	2.68 3.04 2.30 3.00	1.98 1.75 1.49	0.60 0.48	0.30 0.34	2.11	1.40 1.68 1.66	1.60 1.56 1.20	1.18 1.30 1.08	0.70 1.00 1.15
	30 MIN	4.00 4.78	4.20 4.32 4.48	3.60 3.04 4.30 4.30	4.78 4.81 4.90 5.00	4.81 4.90 5.26 5.08	5.09 4.92 5.08 5.00	5.15 5.05 5.45 5.36	4.91 5.08 4.11	3.33 3.37 3.11	1.78 1.90 1.90	1.20 1.30 1.30	2.89 3.06 3.11	3.15 3.15 3.50	3.12 3.15 3.83	2.99 3.24 3.00	3.46 3.56 3.32	0.31 0.60 0.40	0.34	0.34	0.34 0.30	0.34 0.30	1.11 1.15	0.90	0.48

TABLE 9. ANOVA FOR BACTERIAL ANALYSIS BEFORE DISINFECTION

Grouping Variable Information

Variable	Number in Group	Values
Wastewater	5	1, 2, 3, 4, 5
Disinfectant	2	ClO_2, Cl_2
Disinfectant Dose	3	2, 5, 10 mg/l

ANOVA — Log [N (t)] Dependent Variable*

	Degrees of Freedom	Sum of Squares	F-Value	Probability F F-Exceeded
Wastewater	4	13.2259	80.64	0.0001
Disinfectant	1	0.0002	0.00	0.9516
Disinfectant Dose	2	0.1625	1.98	0.1483

* Interactions omitted; N (t) = Number of bacteria at time t.

effect on the number of surviving bacteria. (These effects will be examined in subsequent ANOVA's). This implies that the duplicate experiments run on the same wastewater are not significantly different from each other. For this reason, the experiments were no longer grouped according to wastewater, so that variations seen between experiments in subsequent statistical analyses are due to variations in the wastewater. In other words, the wastewater variable is redundant in that the experiments differ only due to the effects of wastewater composition.

The previous statistical analyses have been run with the logarithms of bacterial numbers from each sample (see Tables 7 and 8) as the dependent variable. In order to compare the bactericidal efficiency of chlorine and chlorine dioxide and to examine the relationships of disinfectant doses and contact times to bactericidal efficiency, a survival ratio for each experiment was calculated and the logarithms of these survival ratios were substituted as the dependent variable.

The survival ratio is defined as:

$$N(t)/N(O)$$

Where:

$N(t)$ = number of bacteria measured at time t,

$N(O)$ = number of bacteria measured at time O.

The logarithm of survival ratio is then:

$$\log[N(t)/N(O)] = \log[N(t)] - \log[N(O)]$$

log[N(T)] was calculated as the arithmetic mean of the logarithms of bacterial numbers from each sample (see Tables 7 and 8) for each time t. The log[N(O)] was similarly calculated at time O. This calculation yields the geometric mean of the replicate analyses for bacterial numbers.

Experiment, disinfectant, disinfectant dose, and time were used as independent variables. The resulting ANOVA is shown in Table 11. This ANOVA identifies several sources of variation:

1. There is a significant difference between experiments. This has been discussed above and is due to variations in the wastewater used for the experiments.
2. There is a significant difference between disinfectant doses; i.e., as the dose increases, the survival ratio decreases.
3. There is a significant difference between contact

TABLE 10. ANOVA FOR EXAMINING EXPERIMENTAL REPRODUCIBILITY

Grouping Variable Information

Variable	Number in Group	Values
Wastewater	5	1, 2, 3, 4, 5
Experiment	8	1, 2, 3, 4, 5, 6, 7, 8
Disinfectant	2	ClO_2, Cl_2
Disinfectant Dose	3	2, 5, 10 mg/l
Contact Time	3	5, 15, 30 min

ANOVA — Log [N (t)] Dependent Variable*

Source	Degrees of Freedom	Sum of Squares	F-Value	Probability F-Exceeded
Wastewater	4	40.3664	90.86	0.0001
Experiment	3	0.5532	1.66	0.1735
Disinfectant	1	7.5686	68.15	0.0001
Disinfectant Dose	2	781.5436	3518.42	0.0001
Contact Time	2	48.2474	217.20	0.0001

* Interactions omitted; N (t) = Number of bacteria at time t.

TABLE 11. ANOVA FOR COMPARISON OF CHLORINE AND CHLORINE DIOXIDE AS BACTERICIDES

Grouping Variable Information

Variable	Number in Group	Values
Experiment	8	1, 2, 3, 4, 5, 6, 7, 8
Disinfectant	2	ClO_2, Cl_2
Disinfectant Dose	3	2, 5, 10 mg/l
Contact Time	3	5, 15, 30 min

ANOVA — Log [N (t)/N (0)] Dependent Variable*

Source	Degrees of Freedom	Sum of Squares	F-Value	Probability F-Exceeded
Experiment	7	21.8691	27.83	0.0001
Disinfectant	1	4.8584	43.28	0.0001
Disinfectant Dose	2	279.2971	1244.08	0.0001
Contact Time	2	37.3407	166.33	0.0001
Disinfectant - Disinfectant Dose Interaction	2	1.3392	5.97	0.0037
Disinfectant - Contact Time Interaction	2	6.4239	28.61	0.0001
Disinfectant Dose - Contact Time Interaction	4	3.4030	7.58	0.0001

* Other interactions omitted; Log [N (t)/N (0)] = Log [survival ratio].

TABLE 12. EXPERIMENTAL DOSE-TIME MATRIX FOR CHLORINE—CHLORINE DIOXIDE COMPARISON WITH MEAN LOG [SURVIVAL RATIO]* AND RESIDUALS

	Chlorine Dose mg/l			Chlorine Dioxide Dose mg/l		
Contact Time	2	5	10	2	5	10
5 Minutes	-0.49 *(1.36)*	-2.23 *(4.03)*	-3.44 *(8.64)*	-1.34 *(.66)*	-3.01 *(2.05)*	-4.40 *(6.33)*
10 Minutes	-1.38 *(1.28)*	-2.98 *(3.88)*	-4.62 *(8.47)*	-1.43 *(.58)*	-3.69 *(1.44)*	-5.09 *(5.48)*
30 Minutes	-2.18 *(1.18)*	-3.72 *(3.78)*	-5.53 *(8.08)*	-1.61 *(.49)*	-3.50 *(1.23)*	-5.81 *(4.87)*

*Calculated means of 8 experiment replicates.

times; i.e., as the contact time increases, the survival ratio decreases.

4. There is a significant difference between the disinfectants, chlorine dioxide and chlorine. The survival ratio overall is lower for chlorine dioxide than for chlorine.
5. There is a significant *interaction* between disinfectant and disinfectant dose; i.e., the relationship between log [survival ratio] and dose is not the same for both disinfectants.
6. There is a significant *interaction* between disinfectant and contact time; i.e., the relationship between log [survival ratio] and contact time is not the same for both disinfectants.
7. There is a significant *interaction* between disinfectant dose and contact time; i.e., the relationship between log [survival ratio] and contact time is not the same for all doses.

In Table 12, the means of log [survival ratio] for chlorine dioxide are compared with that of chlorine by dose and contact time. These means were calculated over all eight experiments. Mean residual concentrations are shown in parentheses. The means of Table 12 show the differences between chlorine and chlorine dioxide observed in the analysis of variance main effects of Table 11. (Observation 1 from above has

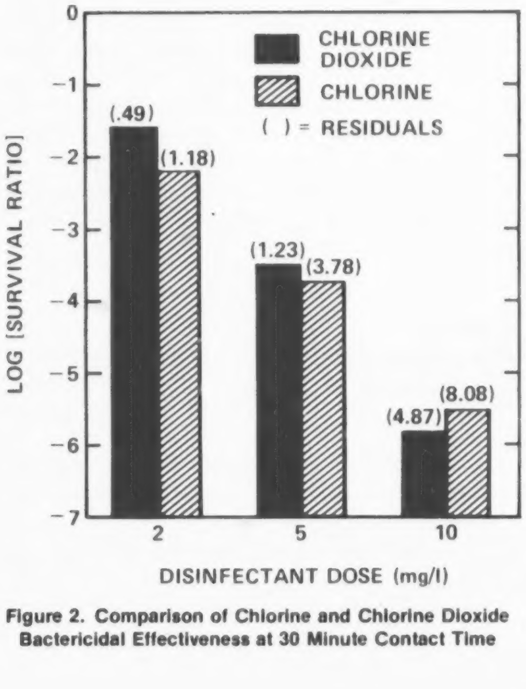
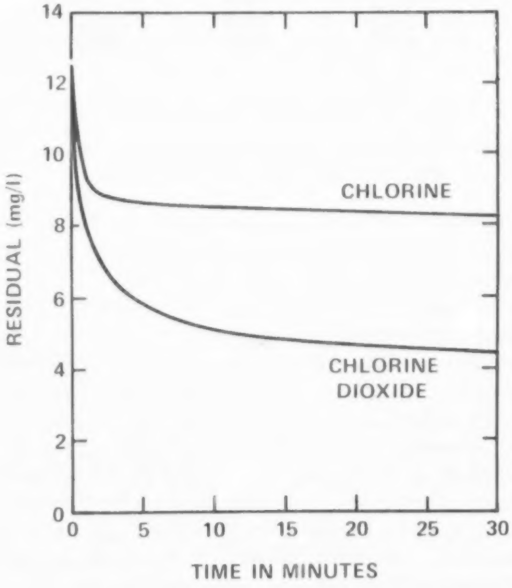


Figure 2. Comparison of Chlorine and Chlorine Dioxide Bactericidal Effectiveness at 30 Minute Contact Time



INITIAL DOSE OF 12.5 mg/l FOR BOTH CHLORINE AND CHLORINE DIOXIDE

Figure 3. Chlorine and Chlorine Dioxide Die-Away in Palo Alto Secondary Effluent

already been discussed.) From Table 12, the difference between disinfectant doses (observation 2) is obvious, as in the difference between contact times. The difference between disinfectants seen in the ANOVA of Table 11 (observation 4) is modified by the ANOVA interactions (observations 5-7). To examine the difference between disinfectants and their relationships to dose and contact time, Figures 2, 3 and 4 are presented.

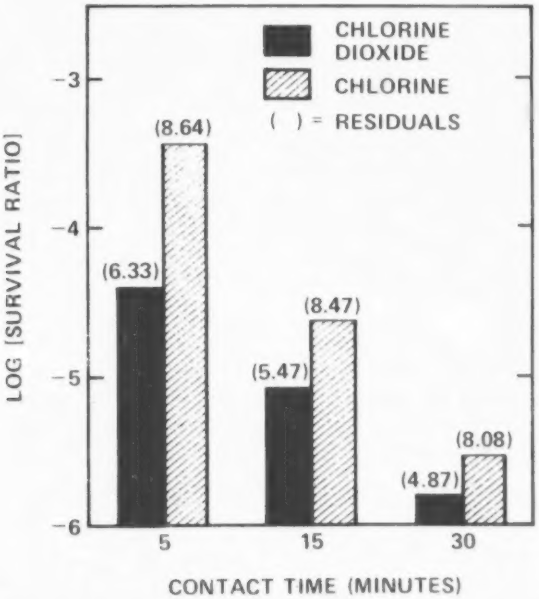


Figure 4. Comparison of Chlorine and Chlorine Dioxide Bactericidal Effectiveness at 10 mg/l Dose

Chlorine and chlorine dioxide are compared by dose at 30-minute contact times in Figure 2. These values are means over eight experiments. The values in parentheses are the means of the residual over the eight experiments. Chlorine gives a lower log [survival ratio] at the 2 and 5 mg/l doses, while chlorine dioxide gives a lower log [survival ratio] at 10 mg/l dose. This is the disinfectant dose interaction mentioned above (observation 5). The reason for this dose dependent disinfectant difference can be found in the die-away curves of chlorine and chlorine dioxide. Figure 3 shows typical chlorine and chlorine dioxide die-away curves in secondary effluent. The initial oxidant demand (within the first three minutes of contact time) is much greater for chlorine dioxide than for chlorine. This initial oxidant demand has been shown by others *not* to contribute to the destruction of bacteria but is a function of waste composition. The disinfection process proceeds after an initial time lag and is dependent on the concentration of bactericidal agent present (4, 14). If, as in the case of a 2 mg/l dose of chlorine dioxide, the initial demand reduces the disinfectant concentration significantly, the rate of disinfection will decrease.

When disinfectants are compared with respect to contact times, the disinfectant-contact time interaction (observation 6) can be evaluated. In Figure 4 chlorine and chlorine dioxide are compared by contact time at 10 mg/l dose. This figure indicates a different time-dependent mode of action for chlorine versus chlorine dioxide. Chlorine dioxide affects a major portion of bacteria destruction at shorter times, while chlorine requires longer contact times to effect the same degree of disinfection.

The disinfectant dose-contact time interaction (observation 7) is a combination of the disinfectant-dose interaction and the disinfectant-contact time interaction and indicates that the rate of bacterial destruction is dependent on both the concentration of the disinfectant present and contact time.

A final analysis of variance was computed to assess the effect of residual on log [survival ratio]. In this ANOVA, disinfectant dose grouping variable was replaced by residual concentration as a co-variate. Also, experiment grouping variable was replaced by total filterable residue, pH and COD, as co-variates in an attempt to isolate the experimental variation. The results of this ANOVA are reported in Table 13. As seen from this ANOVA, residual concentration has a significant effect on log [survival ratio]. Also, total filterable residue has a significant effect on log [survival ratio]. This relationship is such that as total filterable residue increases, log [survival ratio] increases. This observation supports the finding that solids protect bacteria from attack by bactericidal agents.

Conclusions

The conclusions concerning the comparative bactericidal action of chlorine and chlorine dioxide derived from the the statistical analysis are as follows:

1. **Both chlorine and chlorine dioxide give decreased survival ratios when dose or contact time is increased.**
2. **Although some variations exist, chlorine and chlorine dioxide give essentially the same survival ratios when compared on a *mass dose* basis at 30 minute contact time.**

TABLE 13. COMPARISON OF CHLORINE AND CHLORINE DIOXIDE AS BACTERICIDES WITH RESIDUAL CONCENTRATION AS CO-VARIATE

Grouping Variable Information

Variable	Number in Group	Values
Disinfectant	2	ClO_2, Cl_2
Contact Time	3	5, 15, 30 min

ANOVA — Log [N(t)/N (0)] Dependent Variable

	Degrees of Freedom	Sum of Squares	F-Value	Probability F-Exceeded
Disinfectant	1	4.8584	12.02	0.0007
Contact Time	2	37.3407	46.19	0.0001
Residual Concentration	1	242.8966	600.88	0.0001
Disinfectant-Contact Time Interaction	2	6.4239	7.95	0.0005
Disinfectant-Residual Interaction	1	8.8487	21.89	0.0001
Total Filtrable Solids	1	15.2176	37.65	0.0001
pH	1	0.8702	2.15	0.1447
COD	1	0.0156	0.04	0.8448

*Log [N (t)/N (0)] = Log [survival ratio].

3. **Chlorine dioxide is a more rapid disinfecting agent, effecting greater bacterial destruction than chlorine at the shorter contact times.**
4. **Comparing chlorine and chlorine dioxide on a residual basis, chlorine dioxide effects the same survival ratio as chlorine but with a much lower residual concentration.**

DISINFECTION MODEL FOR CHLORINE AND CHLORINE DIOXIDE

To assimilate the information of the previous sections into a useful tool for the design of chlorine dioxide disinfection facilities for wastewater, a simple model was devised which is applicable to both chlorine and chlorine dioxide disinfection. The original form of this model was proposed by Gard (5) to describe the inactivation of viruses by chemical agents. This model has been modified by other researchers, and used to describe disinfection by chlorine (14). Gard found that the relationship between time and virus inactivation describes a linear function on a log-log plot. He also argued that toxicant concentration and time had the same relation to inactivation, and therefore could be used interchangeably as the product ct, where c is concentration at time, t. Gard also recognized that this model was not limited to the polio virus-formaldehyde system which he had studied.

The model which follows is a simplification of Gard's original model. When survival ratio is plotted versus the residual-time product on a log-log plot, a straight line relationship results. Survival ratio was calculated as previously, and residual-time is the product of the amperometric back-titration residual measured at time t, and the sampling time, 5, 15, or 30 minutes. A linear regression of log [survival ratio] on log [residual-time] yields an equation of the form:

$$N(t)/N(O) = [b(RT)]^k$$

where

$N(t)/N(O)$ = survival ratio
RT = residual-time product, $(\text{mg-min}/l)^{-1}$
b = lag coefficient (mg-min/l)
k = velocity coefficient

The coefficient b is a relative measure of the lag period between dosing and the onset of bacterial destruction; the larger the value of b, the shorter the lag time. The coefficient k, is a measure of the rate of kill; the larger the *absolute* value of k, the faster the kill. Figures 5 and 6 show the relationship between survival ratio and residual-time for chlorine and chlorine dioxide. The equations given are the least-squares line of best fit, and r is the correlation coefficient.

Both curves are plotted in Figure 7 to allow a comparison of the disinfectants. The rates of coliform kill are essentially the same for both disinfectants (k values). The major difference between chlorine and chlorine dioxide is in the lag coefficient, b. This implies that for

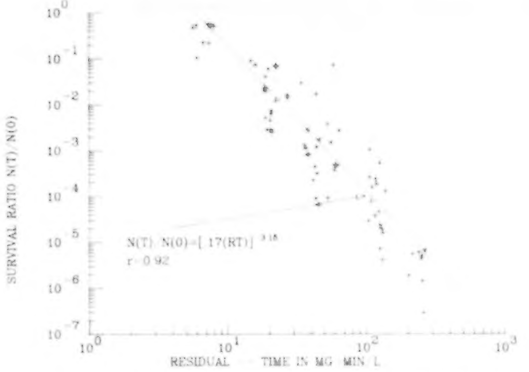


Figure 5. Reduction of Coliform Bacteria by Chlorine

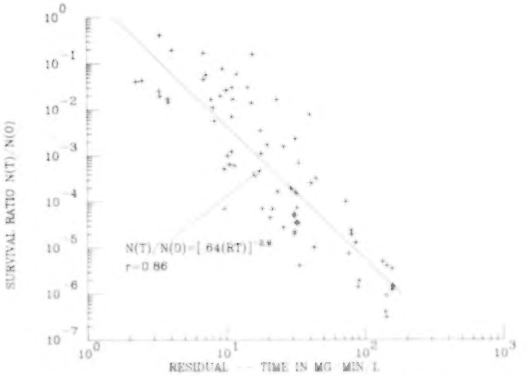


Figure 6. Reduction of Coliform Bacteria by Chlorine Dioxide

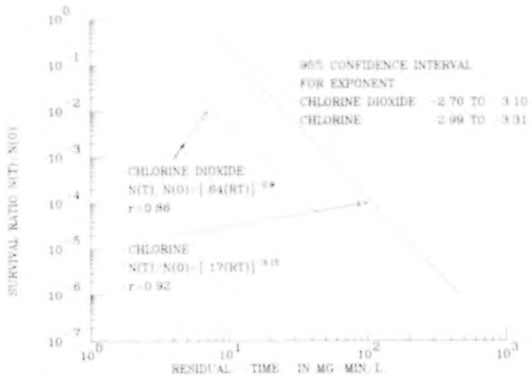


Figure 7. Comparison of Coliform Reduction

equal coliform reduction, a higher residual-time product is required by chlorine than by chlorine dioxide.

Although these results are from only one wastewater, Palo Alto Water Pollution Control Plant secondary effluent, they agree very well with chlorine disinfection results reported by other researchers on a variety of wastewaters.

White (14) reports a b value of 0.23 and a k value of -3 when primary effluent is disinfected with chlorine. Selleck and Collins (4) report b values of 0.25 for a stirred batch reactor and 4.2 for a plug flow reactor and k values of -3 for both in the disinfection of primary effluent with chlorine. In both White, and Selleck and Collins the chlorine residual concentration is *not* the residual as measured at time t, but the residual measured after the immediate chlorine demand has been satisfied. Using the residual concentration measured in this way, as opposed to residual concentration measured at time t, will increase the lag coefficient b. Also, both residual concentration and lag coefficient are functions of wastewater composition and reactor design and comparisons should be made with this in mind.

The most interesting comparison of the work reported here with the work of others is the agreement of the velocity coefficient k. The work reported here measured only total coliform bacteria while in the work of Selleck and Collins, both total and fecal coliform organisms were measured. This comparison indicates that the rate of kill will not be a function of the composition of natural populations of coliform bacteria.

Application of Disinfection Model for Design

To be a useful design tool, a laboratory model derived from batch disinfection studies must have verification from continuous disinfection of wastewater in situations analogous to those found in actual use. In evaluating the model presented here, the basic principles of good disinfection practice must be adhered to: namely, effective rapid mixing of the disinfectant at the point of dose and close approximation to plug flow in the contact zone. If these conditions are satisfied, then the results of batch disinfection studies can be used with confidence in designing disinfection facilities for full-scale operations.

The attractiveness of the proposed model as a design tool rests in its simplicity. It is only necessary to estimate the coliform concentration of the waste stream and calculate the required survival ratio to meet the effluent requirements. Then if the equations given in Figure 7 are solved for RT, it only remains to choose a contact time and corresponding residual concentration. There is, however, an important aspect of the residual disinfectant concentration that must be considered when applying this model to full-scale operations. The laboratory residual concentrations were determined at the specified contact time, 5, 15 or 30 minutes. Most installations, however, monitor and control the disinfectant concentration in the waste stream some short distance downstream of the injection and mixing point but before the stream enters the contact chamber. The measurement of disinfectant at this point in the process does not correspond to the residual measurement upon which the model is based. It does, however, measure disinfectant concentration after the initial rapid oxidant demand of the wastewater discussed previously (see Fig. 3). In the case of chlorine, the die-away curve after this initial demand has a relatively flat slope, so that severe deviation from the chlorine model is not expected. In the case of chlorine dioxide, however, the slope is not nearly so flat (see Fig. 3) so that measurement of the chlorine dioxide residual at the entrance to the contact chamber may substantially overestimate the effective residual concentration. The method of residual measurement must also be taken under consideration when using this model. Most commercial units available utilize continuous flow amperometric determination. As long as these units are calibrated using the amperometric back-titration procedure at pH 7 for chlorine dioxide, design assumptions based on the proposed model will be met. If other than amperometric residual measurement techniques are used, the design procedure must be modified accordingly.

Another aspect of the proposed model that requires verification is its applicability to other wastewater. Although chlorine results reported here correlate very well with the chlorine results of others, it remains to be seen if the chlorine dioxide model will adequately predict the disinfection of other secondary effluents and other effluents of varying quality.

In addition, the economic implications of the proposed model need to be considered. It is obvious from Figure 7 that a shorter contact time is required for disinfection when chlorine dioxide is used as opposed to chlorine at the same residual concentration. Therefore, a smaller contact chamber, or possibly no contact chamber at all, would be required and a savings in capital costs could be realized. However, this savings may be offset by the higher chemical costs of chlorine dioxide (on a mass basis) and a higher chlorine dioxide dose required than that required for chlorine to produce the same residual concentration.

Verification of the proposed model as a design tool entails pilot plant studies (presently in the planning stage) in which the disinfectant is generated and dosed continuously over a period sufficiently long to approximate quasi-steady-state behavior. In addition to confirming the validity of the model as a design approach, the pilot-plant study will provide independent data on the yield and purity of the chlorine dioxide generated by alternative processes.

The pilot plant will be assembled at the Palo Alto Water Pollution Control Plant. This facility is a non-nitrifying, activated-sludge plant with a capacity of 35 mgd. The laboratory-scale studies from which the model was formulated were conducted with this secondary effluent. The pilot plant will be a 10 gpm facility with both chlorine dioxide and chlorine feed systems. This will allow a direct comparison of chlorine dioxide and chlorine.

The feed systems for both the chlorine dioxide and chlorine will be commercially available units or scaled-down versions of these units. Ideally, the chlorine dioxide generator will provide chlorine dioxide free of chlorine and chlorite (i.e., the acid activation of chlorite method of generation). It remains to be seen if units utilizing this process are available for treatment of only 10 gpm, or if design and manufacture of a special unit for pilot-plant studies will be required. Manufacturers of chlorine dioxide generators have indicated that yield and purity may suffer when attempting to scale down units designed for much higher flows. Within the constraints imposed by small-scale operation, information on yields and purity of the chlorine dioxide will be obtained that will be useful in evaluating alternative generation processes.

The contact chambers for pilot-plant studies will be plug flow units. Two types of reactors approximating plug flow conditions can be used: 1) a baffled rectangular tank; 2) a closed conduit. If possible, both types of reactors will be constructed and compared. The rectangular tank contactor will have removable baffles so that the length-to-width ratio may be varied to determine optimum contactor configuration. The contactors will provide 30-minute contact time, and have sample ports placed at 15-minute detention intervals along the reactor. Because initial mixing is an important factor in rate of disinfection, a high turbulence in-line mixer or batch mixer will be used immediately following chemical addition.

Influent concentration of chlorine dioxide to the contactors will be analyzed on a continuous basis. The feed rate will be controlled by the output signal of the continuous monitor. Chlorine concentration will also be monitored and controlled by this method, when chlorine is being fed. Commercial units are available for this type of control for chlorine only. However, these units are based on amperometric measurements and should be readily adaptable to chlorine dioxide measurements.

The pilot plant work described above will provide the information needed for any modification of the proposed model. In addition, data will be gathered on the yield of chlorine dioxide from several generation processes in order to provide guidelines for the design and economic analysis of full-scale disinfection facilities utilizing chlorine dioxide.

SUMMARY

What has been reported in this paper are the results of comparative chlorine and chlorine dioxide disinfection studies. These studies were carried out in municipal secondary effluent and the organisms used as a measure of disinfection were the naturally present population of coliform bacteria. Every effort was made to simulate an actual disinfection process on a laboratory scale. The results of these experiments were developed into a simple disinfection model. Subject to full-scale verification this model can serve as a guideline in the design of disinfection facilities utilizing chlorine dioxide. The rapid destruction of bacteria and virus (14) by chlorine dioxide indicate that chlorine dioxide is a very promising candidate to replace chlorine in wastewater disinfection. The economic evaluation of chlorine dioxide in the disinfection of municipal wastewater is continuing under this EPA research grant.

ACKNOWLEDGEMENT

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DISCUSSION

MR. MECKES: Mr. Aieta and Mr. Berg will now entertain questions from the floor.

DR. LONGLEY: I would like to ask a question of the first speaker . . . in fact, I have a couple of questions. You mentioned the membrane filter technique as being preferable to the MPN. I am curious whether you are using the one-step procedure or the two-step enrichment procedure.

MR. BERG: For the membrane filter technique, we used a direct one-step technique. We did not enrich.

DR. LONGLEY: At the University of Texas, San Antonio, we are doing very similar work, and it is ongoing, comparing chlorine and chlorine dioxide. In general, the results that you discussed here and presented here today correspond very closely to what we are seeing also, particularly in relation to the coliphage. The big difference is, though, that on the fecal coliform, we are finding on a mass basis of applied chlorine or chlorine dioxide a much more rapid fecal coliform kill, and a much greater fecal coliform kill with chlorine dioxide than with chlorine. We are using fecal coliform . . . we are not looking at total coliform and we are using the same dose of disinfectant as you are, that is, 5 mg/l. These results will be presented in about a week and a half at WPCF in Anaheim.

QUESTION: Are we making the ClO_2 the same way, that is, sodium chlorite with hydrochloric acid?

DR. OLIVIERI: I feel like my thunder has been stolen. Unfortunately, Dr. Longley has reported on his studies, and I would like to just indicate that we agree with the virus data that you have. Apparently, the bacteriophage are very nice little systems with which to evaluate disinfectants. In any event, we find the same thing. In pilot plant studies at 10 gpm with secondary trickling filter effluent, we find that the chlorine dioxide performs much, much better than chlorine, with several orders of magnitude difference between the inactivation of total coliform done by the most probable number procedure, compared with chlorine. Do you have any response or any reasoning why the coliform kill in your studies is so much lower than many of the other studies reported in the literature? This, by the way, is an extension of Walter's work that was done at Hopkins.

MR. BERG: By using both counting methods, we achieved comparable results, typically two to three log reductions at a 5 mg/l dose, and five minute contact time.

DR. OLIVIERI: I am talking about differences between chlorine and chlorine dioxide of 2 to 3 mg/l. The overall kill is about five log reduction, bringing the level of total coliforms well below a hundred.

MR. BERG: Okay. I am substantiating our results in that, by both counting methods, we did get two to three log reductions, and that was consistent throughout the study. Why we never exceeded that inefficient log kill, compared with what we see in the literature, I tried to address in the second part of the Paper. The only response I have to that is, we compared pure cultures in native populations.

DR. OLIVIERI: These are natural populations, and I think in Karl's study they are also natural populations. In the studies of O'Brien and Gier, using a combined wastewater overflow, they are also natural populations of coliforms.

MR. AIETA: And you are showing a two or three log reduction . . .

DR. OLIVIERI: Difference between chlorine and chlorine dioxide . . . an overall four and a half to five log reduction, with comparable dosages.

MR. AIETA: Is this on a lab scale?

DR. OLIVIERI: This is pilot plant, 10 gpm.

MR. AIETA: I do not have any explanation at all. We have seen other reports in the literature. All we can say is, on Palo Alto's secondary effluent in our laboratory reactor, we are showing chlorine and chlorine dioxide doing about the same job, as far as coliforms go. We have not gotten to our pilot plant work yet, so the situation may change there. I really do not have any insight into the difference.

DR. RICE, *Jacobs Engineering:* You had said that you are going to be looking at the effects of ClO_2 synthesized other ways. Have you decided which ways you are going to be synthesizing ClO_2?

MR. AIETA: The units that are available to us utilize, essentially, the same reaction that we generate our chlorine dioxide with here. The only difference is that we strip the chlorine dioxide out of solution, so that we are sure that we have a pure chlorine dioxide solution to deal with. I know in the chlorine/chlorite procedure, unless it is controlled very carefully, you can have your stream contaminated with chlorine. I call it contaminated, because what you are looking for is pure chlorine dioxide. The kind of things we would be looking at are what effects the generation procedure, in a natural disinfection scheme, might have on disinfection.

MR. PETER DeSTEFANO, *Riddick and Associates:* I understand that chlorine dioxide is available commercially in a stabilized form in a five percent solution, as active ClO_2. I was wondering if you looked into that at all?

MR. AIETA: No, we have not.

MR. DeSTEFANO: Could you give us just a hint of what the costs of ClO_2 are, as compared to chlorine?

MR. AIETA: I really do not know, but I think there may be some people in this audience who can give you a hand on that.

DR. SUSSMAN, *Olin Corporation:* On the question of stabilized chlorine dioxide, be careful. The solution to the best of our knowledge is a solution of sodium chlorite. If you have worked at all with chlorine dioxide, you will know that it is colored . . . if you get even a very dilute solution, you have a yellow solution. If you look at the stabilized chlorites that are available, they are colorless.

MR. FLUEGGE, *Carborundum Company:* Mr. Berg, in looking at your virus work, you, evidently, were using poliovirus I seed as your virus, as opposed to naturally occurring virus. Do you have any indication of the background virus levels in the water?

MR. BERG: Zero time samples were taken before inoculation of the poliovirus and no recoveries were made. In fact, that is one of the reasons we went with the phage as an *in situ* virus, because poliovirus was very hard to recover.

MR. FLUEGGE: What were the levels of the poliovirus I seed . . . what concentrations?

MR. BERG: At about 10^6/100 ml.

11.

EFFECT OF PARTICULATES ON INACTIVATION OF ENTEROVIRUSES IN WATER BY CHLORINE DIOXIDE

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ABSTRACT

The ability of suspended matter and viral aggregation to affect disinfection efficiency assumes importance in wastewater treatment. Reduced reactivity of chlorine dioxide ($\mathrm{ClO_2}$) *to form carcinogenic compounds is known, but information is needed about the disinfecting ability of* $\mathrm{ClO_2}$ *as affected by particulates and viral aggregates. In laboratory studies at 5° C and pH 7, poliovirus 1 preparations containing mostly viral aggregates took 2.7 times longer to inactivate with* $\mathrm{ClO_2}$ *than single state virus preparations containing 93% single and 7% clumped viruses, as determined by electron microscopy. The disinfection efficiency of* $\mathrm{ClO_2}$ *with unassociated poliovirus 1 singles increased as the temperature increased from 5 to 15 to 25° C at pH 7. However, the disinfection efficiency of* $\mathrm{ClO_2}$ *with bentonite-adsorbed poliovirus 1 singles decreased with increasing temperature relative to the efficiencies obtained with unassociated poliovirus 1 singles. Protection from inactivation due to the bentonite was found to be 11.4% for turbidities of* $\leq$ *5NTU's (Nephelometric Turbidity Units) and 24.8% for* $> 5 \leq 17$ *NTU's. Poliovirus in association with BGM (Buffalo Green Monkey) tissue culture cells show no trend towards cellular protection at pH 7.0 in a turbidity range of 1.10 to 2.00 NTU and 5° C, or in 1.12 to 3.10 NTU at 25° C. Thus, temperature and the amount of turbidity affect the rate of inactivation of bentonite-adsorbed poliovirus, while there is no affect seen with the turbidity levels of cellular-associated virus examined.*

INTRODUCTION

It has been amply demonstrated in recent years that the majority of viruses in the natural environment are associated with solids and are not in a "free" state (Berg, 1973) (2). Wastewater influent, effluent, and chlorinated effluent samples have been found to have 16.1 to 100% of their total virus content associated with solids (Wellings *et al.*, 1975) (32). The ability of viruses to associate with clays and other suspended solids in water without loss of infectivity has been well documented (Schaub *et al.*, 1974 (21); Schaub and Sagik, 1975 (20)). It has been also demonstrated that animal viruses (e.g., poliovirus 1 and coxsackievirus A9) are more resistant to inactivation by free and combined forms of chlorine and chlorine dioxide than the pathogenic indicator of fecal pollution, *Escherichia coli* (Scarpino *et al.*, 1972 (18); Scarpino *et al.*, 1974 (19); Cronier *et al.*, 1978 (4)). This fact demonstrates the need to increase our understanding of the effectiveness of the viral disinfection process in wastewater. The capability of coliforms and fecal coliforms to regrow in chlorinated wastewater effluents (Shuval *et al.*, 1973) (24) may necessitate the presence of a disinfectant residual in these effluents. Such residuals should be non-toxic to aquatic life. The importance of understanding the disinfection kinetics of viruses in effluents from wastewater treatment is clear, since these effluents discharge into receiving waters which may serve either as a source of potable water, an area in which shellfish may be harvested, or as a recreational resource (Murphy, 1977) (13). Emphasis must also be placed on the products of the chemical and physical interactions and reactions between the disinfectant and wastewater effluent, since these products may ultimately affect the environment and its life forms. Such products of disinfection are chlorinated hydrocarbons. These compounds are formed when disinfectants such as chlorine, bromine, and iodine (Mills, 1978) (11) react with precursor organics in water. Trihalomethanes, which are carcinogens and suspect carcinogens are, however, produced to a lesser extent when wastewater effluent is chlorinated because the high ammonia content of wastewater leads to the formation of the less reactive chloramine species (Mills, 1978). The precursor organics necessary for trihalomethane formation have been demonstrated to be the natural humic material present in virtually all source waters (Stevens *et al.*, 1976 (30), Rook, 1976 (17)). Therefore, disinfectant residuals present in discharged, treated wastewater effluent are of concern since they may react with these precursors in receiving water environment to form trihalomethanes and other substances potentially adverse to humans and aquatic life. Disinfectants such as ozone (Falk and Moyer, 1978) (7) and chlorine dioxide (Miltner, 1976) (12) are not known to form trihalomethanes. Their oxidation products created by reaction with organics in water and wastewater are in low amounts and of still largely unknown toxicological significance. Although disinfectant residuals in wastewater effluents are not available with ozone, they are available with chlorine dioxide. As seen in Figure 1, chlorine dioxide (ClO_2) is just as efficient as chlorine when comparison is made between 99% poliovirus 1 inactivation at 15°C by hypochlorous acid at pH 6 to that of chlorine dioxide at pH 7 (Siders *et al.*, 1973 (25); Scarpino *et al.*, 1974 (19); Esposito *et al.*, 1974 (6); Cronier *et al.*, 1978 (4)). The present study was undertaken to evaluate the effects of particulates on inactivation of enteroviruses by ClO_2.

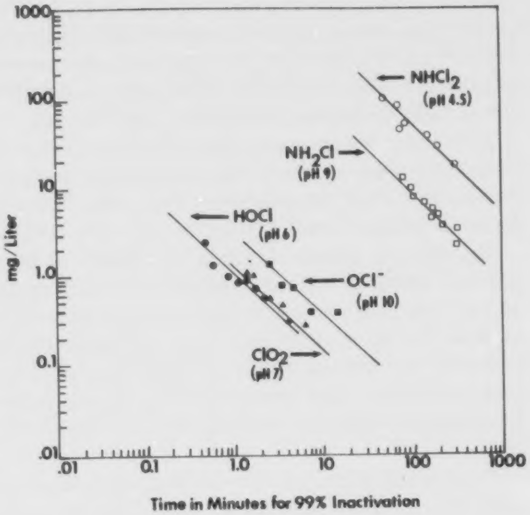

FIGURE 1. Comparison of the Relative Inactivation of Poliovirus 1 by Hypochlorous Acid, Hypochlorite Ion, Monochloramine, Dichloramine, and Chlorine Dioxide at 15°C at Different pH Values.

MATERIALS AND METHODS

Chlorine Dioxide Generation: A stock solution of chlorine dioxide was prepared by the generation of chlorine dioxide gas which occurs according to the following reaction:

$$2ClO_2^- + K_2S_2O_8 \rightarrow 2ClO_2\uparrow + 2K^+ + 2SO_4^=$$

The chlorine dioxide gas was the result of the reaction of sodium chlorite ($NaClO_2$) and potassium persulfate ($K_2S_2O_8$) solutions. The evolved gas was swept from solution by purified nitrogen gas and passed through a column of sodium chlorite to absorb any chlorite gas or volatilized hypochlorous acid that might also be present. Any sodium chlorite dust was retained in an empty vessel prior to collection of the gas in deionized distilled water held at 5°C. The stock solution was prepared prior to experimentation and had a concentration of 500 to 1,000 mg/l after 15 to 20 minutes of generation.

Chlorine Dioxide Measurement: The concentration of chlorine dioxide was determined by the DPD (diethyl-p-phenylene diamine) method of Palin (1974) (14) and as also set forth in the 14th Edition of *Standard Methods for the Examination of Water and Wastewater* (1976) (27).

Virus Quantitation and Characterization: The virus inocula were quantitated and characterized by electron microscopy using the kinetic attachment procedure as described by Sharp (1974) (22), using a JEM 100 B electron microscope (JEOL, Ltd., Tokyo).

Poliovirus Preparation: The poliovirus 1 used in these studies were prepared by two different methods. Poliovirus for both methods were first grown in monolayers of Buffalo Green Monkey kidney cells (BGM).

The chlorine dioxide-demand-free poliovirus preparation, which contained numerous viral aggregates and cellular debris similar to the presumed state of the virus in the natural environment, was prepared by repetitive freezing (to -70°C) and thawing with differential and high speed centrifugation. The 10^8 plaque forming units (pfu)/ml virus was stored at -70°C until used.

Purified chlorine dioxide-demand-free poliovirus with the virions virtually in a "single" state was prepared by freon extraction of poliovirus from intact BGM cells followed by density gradient centrifugation in sucrose. Fractions were collected and examined by electron microscopy. All relevant fractions were pooled and stored at 4°C without any attempt to remove the sucrose (Floyd *et al.*, 1976) (8). The resultant 10^{10} pfu/ml virus preparation contained no cellular debris and consisted of greater than 93% single virions as determined by electron microscopy.

Bentonite Preparation: The chlorine dioxide-demand-free bentonite suspensions with particulates of approximately 2 µm or less used in the turbidity studies was prepared using the method of Stagg *et al.*, (1977) (26). Turbidity was measured in Nephelometric Turbidity Units (NTU) using a Hach 2100 A Turbidimeter.

Poliovirus-Bentonite Complex Preparation: The virus-bentonite suspensions were prepared by allowing the "singles" poliovirus preparation to associate with the bentonite for 1 hour at constant mixing in demand-free phosphate buffer followed by low speed centrifugation to prepare only bentonite adsorbed-virus for use in experiments.

Cell Associated-Poliovirus 1 Preparation: Cell associated-poliovirus 1 was prepared from BGM Monolayers which had been infected 11 hours earlier. The poliovirus-containing cells were harvested and washed repeatedly in demand-free phosphate buffer prior to experimentation. Despite these repeated washings this preparation still maintained a considerable chlorine dioxide demand. Analysis of the cell associated-poliovirus aggregation state by the Multi-Poisson Distribution Model (Wei and Chang, 1975) (31) indicates that the preparation contained a high population of single virions.

Experimental Procedures: The chlorine dioxide disinfection studies were performed using the kinetic (stirred beaker) apparatus (Scarpino, 1972 (18) and 1974 (19)) and the dynamic (flowing stream-rapid mix) apparatus (Sharp *et al.*, 1976) (23).

The kinetic apparatus (Figure 2) consisted of ClO_2 test and control solutions in stainless steel beakers held at the desired temperature in a water bath, and stirred throughout by glass stirring rods connected to an overhead stirring device. Virus was then inoculated into the solutions, and at specific contact time intervals removed and placed in a chlorine dioxide neutralizing thiosulfate solution.

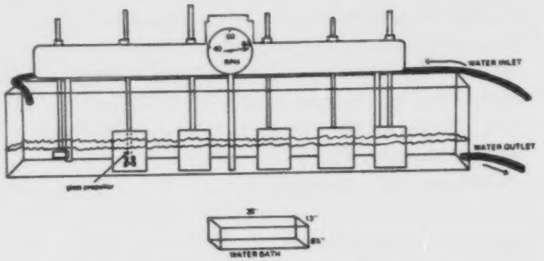

FIGURE 2. Kinetic (Stirred Beaker) Apparatus.

The dynamic apparatus (Figure 3) is ideally suited for yielding kinetic data of short reaction times of less than one minute. The apparatus provides rapid injection and mixing of the virus inoculum into the flowing stream of buffer-containing chlorine dioxide solution. After injection of the inoculum, samples were rapidly withdrawn at the syringe sampling points at appropriate time intervals and immediately mixed with sodium thiosulfate, which was contained within each syringe to stop the reactions. The time of transit of the moving stream of water from the injection point to a sampling

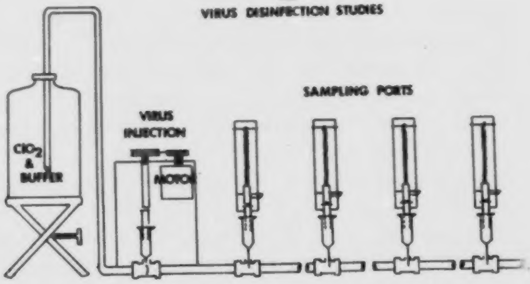

FIGURE 3. Dynamic (Flowing Stream-Rapid Mix) Apparatus.

point was determined by the flow rate and the distance traversed. Turbulent mixing of the inoculum within the stream was assured by maintaining a Reynolds number greater than 3000.

Poliovirus Assay Procedure: The surviving poliovirus from these studies were assayed by the plaque forming system in BGM cells (Dulbecco and Vogt, 1954 (5), Hsuing and Melnick, 1951 (10)). The association of bentonite with poliovirus has previously been shown not to affect the plaque forming ability of the virus (Schaub and Sagik, 1975) (20).

RESULTS AND CONCLUSIONS

A typical disinfection survival curve using the dynamic apparatus and poliovirus 1 singles in contact with 12.1 mg/l chlorine dioxide at 5°C and pH 7 (Figure 4) is obtained by plotting the log of the percent survival against the time of exposure to chlorine dioxide. One hundred percent survival times are derived from the controls. The 99% inactivation points or the 1% survival points were then extrapolated from the survival curves to give the time necessary for 99% inactivation of the viruses. These 1% survival points for each of the ClO_2 levels used were then replotted on log-log paper to show ClO_2 concentration versus the previously determined 99% inactivation times. These new concentration-time plots (Figure 5) were used to compare the rates of inactivation, with the curves closer to the left hand corner of the graph representing the faster reaction. Here we see the effects of temperature on poliovirus inactivation with ClO_2 at pH 7. The 99% inactivation rates of the poliovirus singles are increasing from 5 to 15 to 25°C.

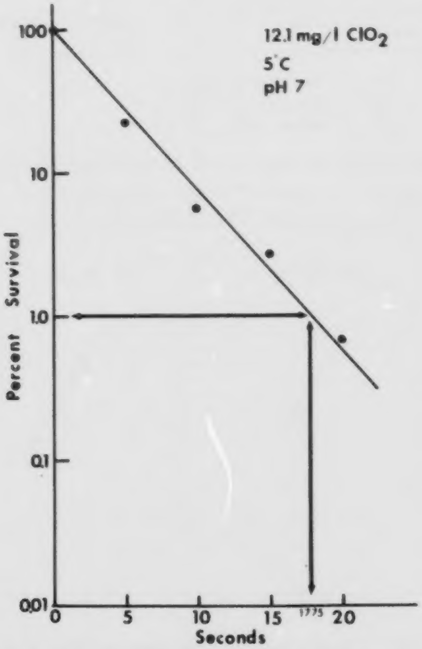

FIGURE 4. Inactivation of Poliovirus 1 Singles Using the Dynamic Apparatus with 12.1 mg/l Chlorine Dioxide at 5°C at pH 7.

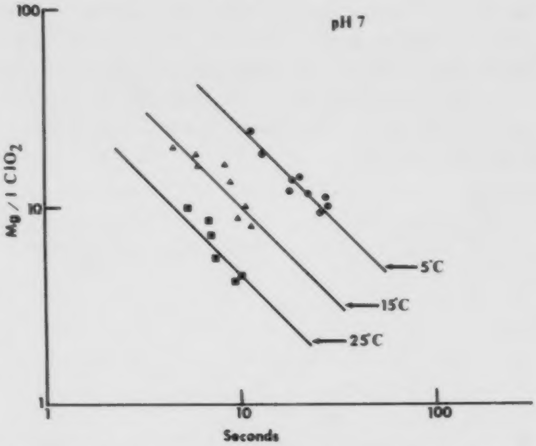

FIGURE 5. Concentration-Time Relationship for 99% Inactivation of Poliovirus 1 Singles Using the Dynamic Apparatus at 5, 15, and 25°C.

The effect of viral aggregation, which is similar to the state of the virus found in the natural environment, was evaluated at 5°C and pH 7 using the kinetic apparatus (Figure 6). The 93% singles poliovirus preparation reached 99% inactivation at a rate of 2.7 times faster than the aggregated virus preparation.

Studies on the effect of inorganic turbidity on the disinfection of bentonite adsorbed-poliovirus I singles were conducted at pH 7 using the dynamic apparatus. The turbidity levels used in these studies fell within the range of expected turbidity levels of final secondary effluent from a well run wastewater treatment plant. The results (Figure 7) for 99% inactivation of free-unassociated poliovirus is represented in the left-hand portion of each graph and the 99% inactivation points for the poliovirus-bentonite complex on the right side for each temperature; the solid line represents the 99% inactivation curve for the free poliovirus. At 5°C there is no trend towards protection from chlorine dioxide inactivation offered by the bentonite to the attached poliovirus with turbidities ranging from 1.14 to 16.5

NTU. At 15°C (Figure 8) a slight trend towards protection from ClO_2 inactivation by the bentonite develops, with turbidity levels that vary from 0.64 to 12.24 NTU. At 25°C (Figure 9) a definite trend toward protection by bentonite is evident where the 99% inactivation points of adsorbed poliovirus at turbidities of from 1.35 to 2.29 NTU are found clustered around the free poliovirus inactivation curve. As the turbidity increases from 3.22 to 14.10 NTU, the 99% inactivation points are found further from the free poliovirus inactivation curve, thereby indicating a definite protective effect at higher levels of turbidity at 25°C. These data indicate that the amount of protection from ClO_2 inactivation by the bentonite to the adsorbed poliovirus increases with increasing temperature and turbidity.

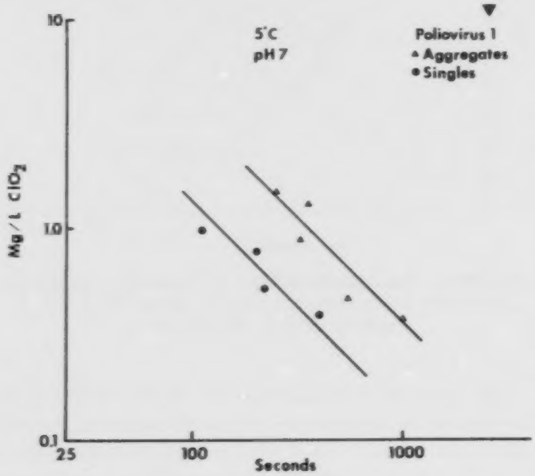

FIGURE 6. Concentration-Time Relationship for 99% Inactivation of Poliovirus 1 Using the Kinetic Apparatus Comparing Single Virions to Aggregated Virions.

When these data are placed into turbidity groupings of ≤ 5 NTU's and > 5 ≤ 17 NTU and graphed along with the unassociated "singles" poliovirus data onto a plot of rate of inactivation versus the product of concentration times temperature a linear relationship results (Figure 10). From this relationship it is found that bentonite-adsorbed virus of the ≤ 5 NTU group is protected to the extent of 11.4% (or 88.6% unprotected) when comparison is made to the unassociated poliovirus. The > 5 ≤ 17 NTU group is protected to 24.8% (or 75.2% unprotected).

Thermodynamic analysis of the data (Table 1) yields mean values for the Q_{10}, Energy of Activation (E_a), Enthalpy of Activation (ΔH), and Entropy of Activation (ΔS) for the unassociated poliovirus, the ≤ 5 NTU group, and the > 5 ≤ 17 NTU group. These values are consistant with those obtained in protein denaturation (Adams, 1959 (1); Cliver *et al.*, 1978 (3); Ginoza *et al.*, 1964 (9); Pollard, 1953 (15); Prokop *et al.*, 1970 (16); Stearn, 1949 (28); and Woese, 1960 (33)). Thus, the information suggests that the mechanism of inactivation for the poliovirus by the chlorine dioxide is by means of protein denaturation. The values for the > 5 ≤ 17 NTU group indicate that the bentonite is interacting with the chlorine dioxide, thus, inhibiting the chlorine dioxide's ability to react with the virus and cause its inactivation.

TABLE 1. Q_{10}, ENERGY OF ACTIVATION (E_a), ENTHALPY OF ACTIVATION (ΔH), AND ENTROPY OF ACTIVATION (ΔS) FOR THE UNASSOCIATED POLIOVIRUS, AND THE ≤5 AND > 5≤17 NTU GROUPS OF THE POLIO-BENTONITE COMPLEXES.

GROUP	Q_{10}	ΔE_a cal/mole	ΔH cal/mole	ΔS cal/mole-deg
Unassociated Poliovirus	2.13	12353	11781	98
≤ 5 NTU Polio-Bentonite	2.35	14020	13448	104
> 5 ≤ 17 NTU Polio-Bentonite	1.56	7289	6717	80

ΔE_a = Energy of Activation
ΔH = Enthalpy of Activation
ΔS = Entropy of Activation

Inactivation studies of BGM cell-associated poliovirus by chlorine dioxide were conducted at pH 7 using the kinetic apparatus. The results (Figure 11) for 99%

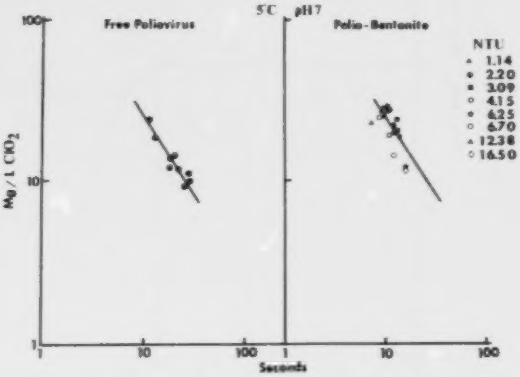

FIGURE 7. Concentration-Time Relationship for 99% Inactivation of Poliovirus 1 Singles and Bentonite Adsorbed-Poliovirus 1 Singles Using the Dynamic Apparatus at 5°C at pH 7.

inactivation of the cell-associated virus are represented here at 5 and 25°C. The solid line at each temperature represents the 99% inactivation curve without data points for the free-unassociated poliovirus (0.15 to 0.20 NTU). Cell-associated turbidities at 5°C of 1.10 to 2.00 NTU and at 25°C of 1.13 to 3.10 NTU show no trend towards protection at these turbidity levels.

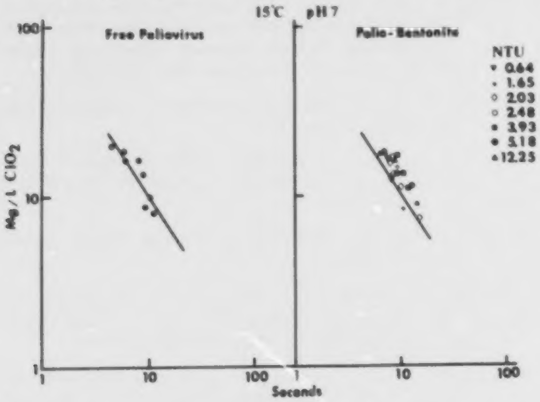

FIGURE 8. Concentration-Time Relationship for 99% Inactivation of Poliovirus 1 Singles and Bentonite Adsorbed-Poliovirus 1 Singles Using the Dynamic Apparatus at 15°C at pH 7.

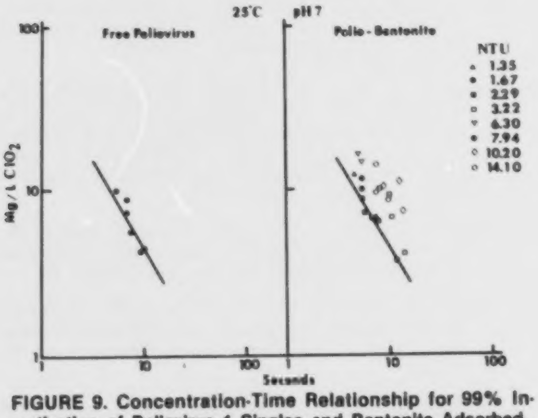

FIGURE 9. Concentration-Time Relationship for 99% Inactivation of Poliovirus 1 Singles and Bentonite Adsorbed-Poliovirus 1 Singles Using the Dynamic Apparatus at 25°C at pH 7.

In conclusion, chlorine dioxide is an excellent disinfectant. Viral inactivation is suggested to be due to protein denaturation. Variations in disinfection rates occur due to viral aggregation even with the same virus type. The effect was clearly seen when the data for aggregated virus, which would be similar to the virus state in the natural environment, is compared to the inactivation rates of the unassociated poliovirus preparation. Thus, this indicates a need for closer examination of time of exposure of the virus to the disinfectant residual to insure proper viral destruction in the effluent. Comparison of chlorine dioxide inactivation of cell-associated poliovirus to unassociated single poliovirus showed no trend towards protection at the turbidity levels examined. This is believed due to the cell associated-poliovirus existing in a "singles" state, and

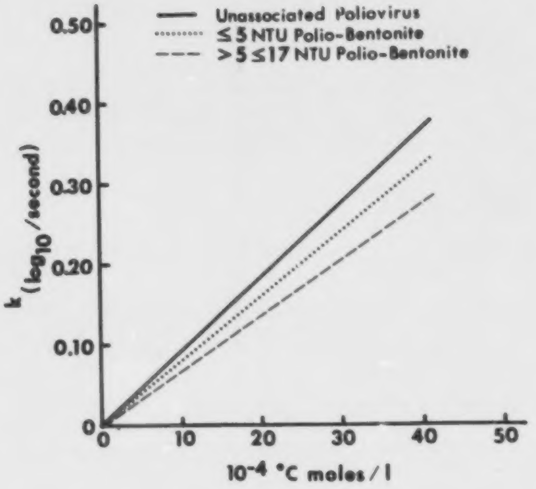

FIGURE 10. Relationship of the Product of Concentration of ClO_2 times the Temperature and the Rate of Inactivation of the Unassociated-Poliovirus, and the ≤5 and 5≤17 NTU Groups of the Poliovirus-Bentonite Complexes.

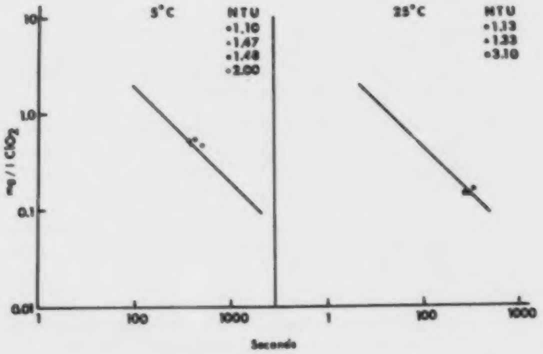

FIGURE 11. Concentration-Time Relationship for 99% Inactivation of BGM Cell-Associated Poliovirus at Various Turbidities Compared to the 99% Inactivation Curve for Unassociated Poliovirus Using the Kinetic Apparatus at 5° and 25°C at pH 7.

that the cellular material surrounding the viruses is readily oxidized by the chlorine dioxide, thus exposing them to the disinfectant as if single virions alone were present. Finally, a correlation exists between bentonite protection of poliovirus during disinfection at increasing temperatures and increasing turbidities, i.e., as the temperature and bentonite turbidity increases, the disinfection efficiency decreases for the bentonite adsorbed poliovirus.

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DISCUSSION

DR. JOHNSON: I was interested in the effect of your picking the 99 percent point for your analysis of data. I noticed the one curve that you showed on the log survival versus time was very linear. Was this typical of your log survival versus time curves?

MR. BRIGANO: Yes.

DR. JOHNSON: They were all linear, is that right?

MR. BRIGANO: Right. These are fitted to a first order reaction, as under the model equation of van't Hoff - Arrhenius.

DR. JOHNSON: Was this true of not only the free unassociated virus, but also the virus which was cell associated and the virus which was associated with the bentonite? You got linear log survival time?

MR. BRIGANO: Yes.

DR. JOHNSON: Surprising. That does not agree with data that we have gotten on aggregation of virus to virus, where we were looking at aggregated virus, rather than virus which was . . . as you are, in your study, associated with cells or bentonite. Our virus-virus aggregation data showed nonlinear log survival versus time plots.

MR. BRIGANO: Log/log plots?

DR. JOHNSON: No, log survival versus time.

MR. BRIGANO: These are log/log plots. The log of the concentration versus the log of the time for 99 percent inactivation.

DR. JOHNSON: Yes, okay. Now, that is my point. If you took, say the 99.9 percent level, or the 99.99 percent level, . . . if your plots of log survival versus time . . . your basic disinfection kinetic data . . . if they are not linear . . . if it does not obey Chick's Law, then where you pick your survival level very much affects the . . .

MR. BRIGANO: Right. I thought you were talking about the van't Hoff plots.

DR. JOHNSON: No, I am talking about the log survival versus time . . . the kinetic plots. Were they linear?

MR. BRIGANO: For the dynamic apparatus, they were. For the kinetic apparatus, they were not. The dynamic apparatus is the second apparatus I showed with the tubing.

DR. JOHNSON: Okay. That is the Sharp, Johnson, Floyd apparatus. With that kinetic data, log survival versus time, if you pick a two log survival level, you get a very different kind of effect, if the survival kinetics are not linear, than if you pick a four or five log survival. My question is, were all of your kinetic data linear?

MR. BRIGANO: Yes, for that apparatus, they were all linear, as shown in that survival curve that I showed in the beginning.

DR. JOHNSON: That is in contrast to our data where they were nonlinear if they were aggregated virus to virus.

MR. BRIGANO: Those were not done using that apparatus for the aggregated virus. The aggregated virus experiments were done in the kinetic apparatus . . . the stirred beaker apparatus.

DR. JOHNSON: Oh, and those log survivals were not linear?

MR. BRIGANO: Right.

DR. JOHNSON: Now, that does make a point, because, if you look at, say your two log survival, that is more associated, because you are taking the initial early part of the log survival curve.

MR. BRIGANO: No, the singles virus was also done. For that graph, where I showed the difference between aggregated virus and the singles virus, they both were done using the kinetic, stirred beaker apparatus. They were not done using the dynamic apparatus. They both were done with all the conditions the same.

DR. KARL LONGLEY: I want to commend you, first of all, on a very interesting paper. Secondly, I want to suggest that it departs quite a bit from what we see at the end of a wastewater treatment plant, where a good part of the viruses are actually occluded in the solids and are not adsorbed, with possibly some exposure externally to the solution containing the wastewater disinfectant. I think, in a real world situation, you may find that the controlling rate parameter is really the rate of diffusion of the disinfectant through the occluding organic material, and the competition of that organic material for the disinfectant, rather than other parameters that might be considered.

MR. BRIGANO: We understand that. Thank you.

12.

PANEL DISCUSSION OF CHLORINE DIOXIDE DISINFECTION

Participants of Chlorine Dioxide Panel Discussion

Mark C. Meckes, Moderator
U.S. EPA, MERL-Cincinnati

1. James Berg
   Stanford University
2. Marco Aieta
   Stanford University
3. Paul Roberts
   Stanford University
4. F. A. Brigano
   University of Cincinnati
5. P. V. Scarpino
   University of Cincinnati

DR. ROSEN: I would like to make these comments, with respect to the Stanford work. Paul Roberts and I were recently at a conference where Dr. Kott was talking about some work that was done at Israel. He found very similar results in an ozone/chlorine comparison to the work that you have reported with chlorine dioxide and chlorine, with respect to the greater relative efficiency of ozone with respect to viruses, in secondary effluents. Now, he did not present data. His comment was from the floor, with respect to someone else's work, so that I cannot comment on the absolute magnitudes. In relative terms, he found a similar relationship to that which you have reported. The other thing that he reported, that I found of interest, that Clifford White is always banging the drum about, is that, really, these problems tend to be sometimes solved by, not one or the other, but by combinations. He reported that, in this case, chlorine and ozone were more efficient (this is, again, relative efficiency, and this was not defined because it was a comment from the floor), in terms of finding some sort of synergism, with respect to both bacteria and viruses on secondary effluents, independent of the method of addition, or the order of addition . . . that is, together, chlorine first, ozone first. Also, with respect to whether it be bacteria or viruses, he found they could get an effect, which was, apparently, synergistic. Perhaps we ought to be talking, in some of these cases, about combined disinfection, maybe something for viruses and something for bacteria, and ending up with something that overall is the most cost effective.

MR. AIETA: Just a comment to go along with that. Hopefully, in our pilot study, we will be able to take a look at the combination of chlorine and chlorine dioxide. Our pilot studies are still pretty much in the planning stage, so we do not know exactly how far we will go with that.

DR. PAUL ROBERTS: I think that may be a good practical suggestion, especially, in that it has been found that combinations of chlorine and chlorine dioxide have the effect of inhibiting, at least haloform formation, and it may be that a good economic combination of chlorine and chlorine dioxide avoids the formation of some of the organics that we would like not to see, and at the same time achieves economic kill of both bacteria and virus.

MR. WHITE: Isn't it because ozone has a different killing mechanism than chlorine?

DR. ROBERTS: I cannot answer that definitively, but it is a good . . .

MR. WHITE: Insofar as bacteria versus virus?

DR. ROBERTS: That is a reasonable hypothesis, I think. Would someone like to comment on that?

DR. ROSEN: I did ask if the work that was done in Israel was with seeded viruses or natural coliphages, and it was with both, and results were similar. That question always arises when you talk about seeding and protection, and some of the work that is coming out of Pat Scarpino's group. Combining disinfectants is something that I am for, and something that I do not think we ought to jump into, maybe, until we understand some of the basics about the individual disinfectants. With respect to the mechanism, I do not know if the mechanisms are the same, but certainly, I think that one indication that the mechanisms are different (there are a couple, in the literature) is, of course, the relative rates of inactivation. The second has to do with the MPN/MF work that has been done, and the fact that, apparently, when that comparison is made with chlorine, there is a different result than when that comparison is made with ozone. That is, ozone seems to make no statistical difference with respect to survivors that may regrow, whereas chlorine does.

DR. RICE, *Jacobs Engineering:* This is a different point. Three points, actually, one is a comment, one is a partial answer, and one is a question. The comment is, so that the audience will not think that the only person hawking a new book here is George White: Those of you who have read ClO_2 literature have run up against a little monograph on the oxides of Cl, published in 1959 by Dunod, written by Dr. Willy J. Masschelein, who is the technical director of the Tailfer Belgium Drinking Water Works. About a year and a half ago, I met Dr. Masschelein, impressed him with a growing amount of interest in the U.S.A. on ClO_2, and talked him into updating his monograph and doing it in English. I have been the editor on it. It is in proof now, and it will be published by Ann Arbor Science Publishers before the end of the year. So, if anybody is interested in that, good. The partial answer, which I have, to the question that was raised a while ago about cost . . . what does it cost to use ClO_2 over Cl_2? That depends pretty much on the way you make ClO_2, and I am not going to get into numbers, but that is why my earlier questions, of how is ClO_2 made, are very important. Again, hawking a book, this is an EPA final report. Some of you may know, I have been involved with a nonprofit organization in Washington, which has been granted by drinking water people here to survey the use of ClO_2 and ozone in drinking water treatment plants. That final report is in print, and in fact, I think it has been shipped from the printer, but it does have a fairly good state of the art review of ClO_2 in drinking water.

Now, back to the cost situation. The people in European drinking water plants that use ClO_2 normally use it as a terminal disinfectant. They normally make it using excess Cl_2 over $NaClO_2$. In some circumstances, it is made by using HCl and the sodium chlorite. These people say that it costs them about three times the amount to use ClO_2, than it does Cl_2. They use very low levels in drinking water. I think, in Zurich it is 0.15 ppm. So, I do not think that any of this reads directly on the wastewater end, but those two comments.

Now, my question is, in the drinking water side, there is some concern by EPA for the toxicity of ClO_2^- ion for people. And, I have not heard any of this yet, in the wastewater discussion. I wonder if there is any testing going on, concerning the toxicity to organisms of the ClO_2^- ion, in the wastewater area?

MR. AIETA: It is my understanding that there is work happening now in the EPA. I think they are feeding chlorite to rats. I have not seen anything come out of that work as of yet. As a corollary to that question, there will be a need to be able to measure chlorite in the presence of chlorine dioxide, at some point in time. That is no easy task.

MR. RICK TRAVER, *EPA, R & D Storm and Combined Sewer Section:* Mr. Berg and Mr. Aieta, in your generation of chlorine dioxide, in your laboratory, how was that accomplished, and to what quantities did you generate it? Was it on a continual basis? Did you store it?

MR. BERG: We generated batches of chlorine dioxide, on the day that we did an experiment, so they were fresh on that day. We took a sodium chlorite 750 ml solution and, I believe it was a ten percent solution of that salt, to which we added a ten percent solution of hydrochloric acid. The ClO_2 that would be evolved from that reaction was then passed through a sodium chlorite salt tower. We have been told that that should remove any chlorine present, although it has also been shown that it is most likely that no chlorine is generated in that process. Then, the gas that was passed through the salt tower was trapped and iced down in deionized water. We then measured the concentrations of that, and, typically, we could get around 500 mg/l.

MR. TRAVER: Was there any storage of the chlorine dioxide, over a period of time, between day to day?

MR. BERG: We stored it, and if we used it later for other studies, we noticed a drop in concentration. We could probably lose up to a 100 mg/l over a few days.

MR. TRAVER: O'Brien and Gier's research was referenced earlier on the utilization of chlorine and chlorine dioxide singly and also combining them, which I have not heard here, either, in tandem in combined sewer overflows, research funded by my particular program. This was one of the problems that was typified in bench scale study, that of storage of the chlorine dioxide solution and the decay of the concentration over a period of time. One of the suggestions was that better results could be found on a continuous full-scale manufactured solution. That was my question . . . a couple of points, though. At the present time, in addition to the research of O'Brien and Gier, a full-scale evaluation is being conducted at the East Chicago Sanitary District in East Chicago, Indiana, coupling a 160 million gallon lagoon, which is receiving secondary plant effluent, and during wet weather 75,000 GPM of combined sewage. A full-scale 16 million gallon chlorine/chlorine dioxide disinfection system is being installed and will be dosing at 8 mg/l of chlorine, two minutes detention time, followed by 2 mg/l of chlorine dioxide, fifteen seconds detention time, attempting to achieve a four to five log decrease, as a result of the O'Brien and Gier study. We will be optimizing that situation, in Chicago, once that goes on line, we hope, later this winter and early spring.

MR. WHITE: Is that the greater sanitary district of Chicago?

MR. TRAVER: No, sir, East Chicago Sanitary District.

MR. WHITE: The answer to the question on cost will be found in the Culp-Wesner-Culp Report, and it is EPA #600/2-78-182, August, 1978. It compares chlorine, onsight generation of hypochlorite, onsight generation of chlorine/chlorine dioxide, chlorine cylinder storage, and ozone. They split it up into three: process energy KW per year, dollars construction cost, and total cost per year, in dollars (thanks to Bob Baker, who took all the tables and put them on log/log paper; I just received them from him, in case this might be asked at the previous panel). This is on potable water, but anybody that wants to get a comparison of costs, can use this very recent report. Now, my question. Mr. Aieta, I am slightly confused about your analytical technique, and I wonder if you would just explain a few things. You talked about iodometric; you talked about amperometric; you talked about different pH's, and you talked about DPD. Now, how do you characterize iodometric, because amperometric is really iodometric back titration, but it is an amperometric end point. So, what is iodometric? And, what were the different pH's? This is what confused me.

MR. AIETA: What I call the iodometric technique is, essentially, the starch iodide. And, that technique for chlorine dioxide is done at a pH below two. The pH characteristically runs about 1.8, 1.85. The amperometric back titration that I spoke of is done at a pH of 7, and at that pH, the chlorine dioxide only goes through a one electron change. The starch iodide method at pH 1.8 should take the chlorine dioxide all the way to chloride. So, that by comparing those two, what I attempted to show was that, if the results did not agree, I would not be anywhere. But, since the results agreed, it showed that, yes, the iodometric is measuring chlorine dioxide, five electron . . . five valance changes. And, the amperometric is measuring one valance change for chlorine dioxide.

MR. WHITE: Okay, so in effect, the amperometric is really the same as the forward titration for free chlorine, right?

MR. AIETA: No, it . . .

MR. WHITE: I am sorry, monochloramine . . .

MR. AIETA: Right, exactly.

DR. OLIVIERI: I would like to direct a question to Mr. Berg and Mr. Aieta. Since we brought up the stock solutions, did you people ever compare the levels of chlorine dioxide, using a direct spectrophotometric measurement at 357 nm with your starch iodide or amperometric measurements? In addition, is there any idea of the percent yield that you got in the preparation of your stock solution?

MR. AIETA: I do not think we ever calculated percent yield on our stock solutions. We never verified our chlorine dioxide spectrophotometrically.

DR. OLIVIERI: I realize that it is not going to work very well in the sewage, but it would certainly work well on your stock solutions.

MR. AIETA: We felt that the starch iodide method was sufficient, and gave us good enough results, so we really did not go any further than that.

DR. OLIVIERI: But, the starch iodide at the low pH does not distinguish between chlorite and chlorine dioxide.

MR. AIETA: Well, in our generation procedure, we feel we have a pure chlorine dioxide solution.

DR. OLIVIERI: The point is, can you demonstrate that?

MR. AIETA: Not with what we have done now, no.

DR. GILBERT GORDON, *Miami University:* We have done work on chlorine dioxide for many years. The direct spectrophotometric analysis is, indeed, a challenge to almost anybody doing it. We developed a shrinking bottle to do that . . . a bottle which has no air volume above the liquid. It is really an inverted syringe, so that as you withdraw solution to analyze it, you are pumping solution out. You are not changing the relative volume above it. Thus, you do not have to worry about the Cl_2 dispersed both in the gas phase and the liquid. Under these conditions, we can reproducibly do several parts per thousand spectrophotometric, and what you are calling iodometric titration. However, we do two added titrations. If you do that iodometric at pH about 8 or 7.8, there is a single electron change. The ClO_2 goes to ClO_2^-, a single electron change. You can then acidify that solution even more and do either a separate or a continued titration, getting the five electron change. And, if you compare those, one can find internal consistency, and check out this point of the purity of the solution . . . the amount of ClO_2^-, or, conversely, if one has stored these at high pH, the problem of chlorate and chlorite formation. In answer to the general question of stability, we have stored solutions for weeks in dark bottles in refrigerators at zero degrees, and find no decomposition. However, the analytical techniques are somewhat trying, and it does take one quite a long time to develop a method where you do not lose gas upon analysis, and are not convinced that you are getting something else. However, on the iodide analysis do you find all your results showed amperometric titrations were higher and iodide were lower as you drew them on the slides, if I recall. I think the numbers you had showed the iodometric values were slightly lower. The amperometric were slightly higher titers, in the titration. I would suggest that may be due to iodate formation, in that, chlorine dioxide reacts with iodide and forms iodate. That iodate does not react very rapidly with iodide to reform iodine. That, quantitatively, can account for the low titer values. We can show those two come out exactly the same, if you do very slow addition, very dilute solution.

MR. BERG: I have a question for you. Have you found, in generating chlorine dioxide by the method we described, that there was any level of chlorite ion in the stock, or was that produced over a time?

DR. GORDON: I think your point is well taken, that if you go through a chlorite purification tower, you do end up with extremely pure chlorine dioxide solutions. We have had solutions which showed no tests for any other foreign material, other than the buffers, in which we may have collected them. There have been several questions about synergistic effects. I think that in the chlorine/chlorine dioxide studies, I would like to suggest a series of what I think are very important potential synergistic tests. If one is adding chlorine dioxide after the chlorine, there is one kind of information. If, however, one injects sodium chlorite, or a solution containing a chlorite ion, I suggest that one might, indeed, observe considerably different results. I am a kineticist, basically, and we have studied these solutions for years. If we look at pure phenol, and look at the oxidation with chlorine, or the oxidation of phenol with chlorine dioxide, or mixtures of those, where we are now injecting phenol with a mixture of the two, one sees considerably different results. We have been able to demonstrate, I think unambiguously on pure solutions initially containing whatever organic material we want to oxidize, and sodium chlorite, to which we add chlorine, that intermediates are formed, and those intermediates are different in the way in which they react with phenols or whatever the organic materials are. If one is looking for synergistic effects, I think one can easily demonstrate it in that case. In the other case, it may be more difficult, because only the chlorite product, after chlorine dioxide reaction, is available for reaction. That synergistic effect may be much further downstream. But, the other one is very easy to demonstrate.

MR. WHITE: The French tell me that the ozone, following the application of chlorine dioxide, puts some of the chlorite form back to chlorine dioxide . . . if you can believe them.

DR. GORDON: The potentials are right for oxidation.

MR. WHITE: They also say, that ozone can reverse the chloride that comes out from, maybe, the prechlorination, back to HOCl.

DR. WAITE, *Northwestern:* I would like to ask if any of the chlorine dioxide people have looked at the reaction of chlorine dioxide with any different types of organics? Realizing the demand of chlorine dioxide is pretty high, in highly contaminated waters, ammonia probably is not going to affect it. Working with a different type of disinfectant, that I am not allowed to mention here, we see that, depending on the type of organic in the water, we see a lot of different reactions, even though the TOC or COD may be the same. So, I would like to ask if any of you have looked at different types of wastewaters, or waters containing a basic TOC but different makeup?

DR. ROBERTS: We have just begun to do so, and I really do not have any results to present. However, I might comment on another finding at the European conference on drinking water treatment, that some of us attended. In Germany and Holland, the use of total organic chlorine, as a collective parameter, is becoming more widespread. Some measurements have been made, particularly, comparisons of formation of total chlorinated organics by chlorine, compared to chlorine dioxide. And, the few data that were shown indicated about one-fourth as much total organic chlorine formed by chlorine dioxide, as for chlorine. In other words, it seems as if those organic chlorine compounds, measured by this method, are formed to a lesser extent using chlorine dioxide than chlorine, which ought to be favorable for chlorine dioxide.

DR. SCARPINO: I just wanted to comment on some other aspect of the research by Mr. Berg and Mr. Aieta. Nowhere in the papers have you mentioned the pH of the system, or the chemical parameters of the system. I may have missed that. What were the pH's in your system?

MR. AIETA: Our pH's were typical of pH's of secondary effluent, i.e., 7 to about 6.5. We measured COD, ammonia nitrogen, alkalinity, and there were a couple of others . . . total suspended solids . . .

DR. SCARPINO: I was especially interested in pH, because the work of Cronier in our lab establishes the fact that as you raise your pH from 4.5 to 9, you increase the efficiency of your chlorine dioxide. This is consistent with other work in the field, but not consistent with the work of O'Brien and Gier, I believe, or Smith from Syracuse.

MR. AIETA: Is that relative to chlorine?

DR. SCARPINO: Chlorine dioxide, yes. I was wondering if you noticed the same thing, as you increased your pH. Did you have a more efficient system?

MR. AIETA: We did run a couple of statistical analyses on our wastewater parameters. The only correlation we could find between log-kill and our parameters was with the solids. As the solids increased, the kill decreased. We found no correlation with pH or any of the other parameters.

DR. SCARPINO: Your pH range was quite narrow.

MR. VENOSA: I would just like to make one comment before adjourning. I am getting the impression here that there is just as much disagreement in the chlorine dioxide field now as there was with ozone several years ago. Everybody is getting controversial. They are getting results that are different from each other. So, it suggests that we need to do a lot more research in this field on chlorine dioxide in both measurement as well as disinfection efficiency.

DR. SCARPINO: I can agree with you on the measurement of chlorine dioxide. I think one of the problems we had, initially, was what method to select for chlorine dioxide. We finally ended up calling Dr. Palen in England to find out exactly how he did it.

SECTION 4. ULTRAVIOLET LIGHT

13.

UTILITY OF UV "DISINFECTION" OF SECONDARY EFFLUENT

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INTRODUCTION

The criterion utilized in these studies to determine the adequacy of the "disinfection" level achieved was a fecal coliform content of 200/100 ml. Fecal coliforms comprise only a part of the total coliform group, and since some members of the coliform group are acknowledged pathogens, we believe that some caution and understanding should accompany the use of the terms "disinfection" or "disinfected". There is also the matter of the Glossary (2) definition of disinfection which excludes viruses, thus reinforcing the need to exercise caution and understanding.

At the onset of this project, the factors that we thought would be among the most important of the many confounding variables (other than the absolute number of fecal coliforms and the ultraviolet dose) were total suspended solids, turbidity, and probably the transmissability at 254 nm. Our subsequent experience taught us that over the range of suspended solids (5-50 mg/l) and turbidity (0.5-12 Ntu) that we encountered, these two quality parameters had relatively little influence on the results. Transmissability, however, emerged as quite important, and furthermore was observed to be a factor of the operation of the biological treatment facility — a complete-mix activated sludge system.

This project was conducted over a 16-month period at the Dallas Water Reclamation Research Center by personnel from Dallas' Water Utilities Department, the Civil Engineering Department of Texas A&M University, and the U.S. Environmental Protection Agency. Detailed descriptions of the Demonstration Plant at this Facility have been published (1). Two ultraviolet (UV) generating units were used (3). First, a Kelly-Purdy unit originally designed for use in shellfish depuration studies (3), and later, a proprietary unit loaned by Ultraviolet Purification Systems, Inc., located in Scarsdale, N.Y. The UV units were applied directly to final effluent, to mixed-media filtered effluent, to dual-media filtered effluent, to tertiary clarified effluent, to chemically clarified effluent, and to tertiary clarified plus dual-media filtered effluent. Additionally, following the protocol used in previous work (5), and in full recognition of its limitations, three virus seeding experiments were conducted using poliovirus type 1 and F2 coliphage.

An attempt was made by the plant operational staff to utilize as a baseline of biological operation the production of a highly nitrified effluent, i.e., ammonia-nitrogen (NH_3-N) effluent concentrations of <1 mg/l as N. We failed all too often to achieve this level because of fragile oxygen transfer systems and a gross operator failure. Actually, three different oxygen-transfer systems were used during the study, two proprietary types and one inadequate home-made type. The operator failure occurred when he left a waste sludge valve wide open all-night long — in the cold month of December.

A stringent sampling program was followed in these studies. The unit processes prior to the UV unit were sampled on a flow-composited basis and analyzed by *Standard Methods* (7) procedures. The UV units were grab sampled at 11 AM and 4 PM — the times when the diurnal curve for organic content was maximum at the Dallas facility. Separate samples were taken for chemical and microbiological analyses. The membrane filter procedure was used with the Kelly-Purdy unit and the MPN procedure with the Ultraviolet Purification Systems unit. Eight paired irradiated samples were run for comparison using both methods. The mean log results were 2.07 by MF and 2.01 by MPN showing essentially no difference between the two.

Kelly-Purdy Unit Results

The Kelly-Purdy (K-P) ultraviolet light disinfection unit (Figure 1) consists of a shallow-tray exposure chamber 185 cm (6 ft.) long, 92 cm (3 ft.) wide, and 9.2 cm (3-5/8 in.) deep. Flow passes through the tray underneath 13 30-watt UV lamps (G30T8). The influent flow was measured with an undulating disc meter, and water depth was controlled by changing the elevation of the effluent weir.

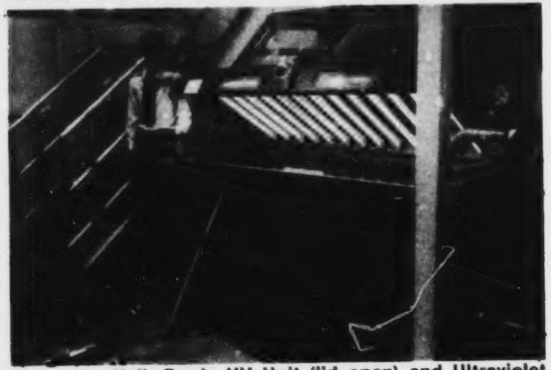

Figure 1. Kelly-Purdy UV Unit (lid open) and Ultraviolet Purification Systems Model EP-50 (on blocks on the floor)

Five runs were made with the K-P unit ranging from 2-1/2 hrs. long to 24 days. The first three runs were applied directly to activated sludge effluent, the last two to a mixed-media filtered effluent. In the first three runs, flow rate was varied from 0.32 to 1.58 l/sec (5-25 gpm) and depth from 2.54 to 5.08 cm (1 to 2 in.). The microbiological results of the samplings showed no association with flow and little with depth, but more importantly, the effluent quality did not achieve the fecal coliform level sought, 200 fecal coliforms per 100 ml. However, the results were definitely promising, giving over-all reductions in fecal coliform concentrations of 2-3 logs. Figure 2, for example, illustrates the results obtained during Run K3. The theoretical detention time for this run was 43 seconds and the theoretical dose 32,000 μwatt-sec/cm^2. Table 1 shows for each run the types of effluent treated, the time and duration of each run, and the average COD and NH_3-N content of the composited samples. For run K3, the effluent composited samples averaged a COD of 43 mg/l and NH_3-N of 1.1 mg/l which indicates good biological treatment — although not quite as good as was sought (NH_3-N $<$1.0 mg/l). COD: BOD_5 ratios for this effluent generally average 2.7:1, hence, a BOD_5 of about 16 mg/l is suggested.

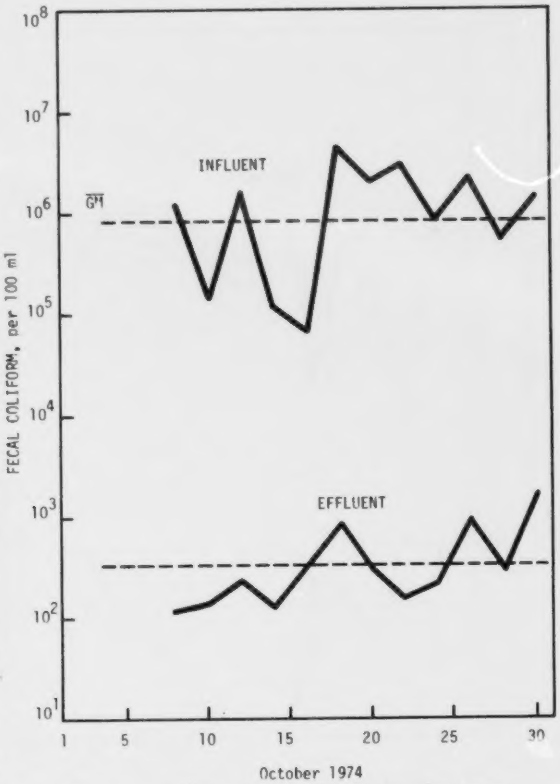

Figure 2. Fecal coliform data for Run No. K3

One of the difficulties encountered with the K-P unit was the accumulation of solids in the tray. Hence, for the last two runs made with that unit, the final effluent was filtered through a mixed-media filter (anthracite-sand-garnet, Neptune-Microfloc Co.) prior to UV irradiation. The microbiological results of Run K4 (Figure 3) show that all samples met the 200/100 ml fecal coliform limit. For Run K5 the flow rate was increased from 0.96 l/sec to 1.4 l/sec (15.25 to 22.2 gpm) thus decreasing the theoretical detention time to 29 seconds. More importantly, however, it was during this run that the operation failure occurred. The result was that only a couple of effluent samples met the fecal coliform limit.

Ultraviolet Purification Systems Unit Results

It was at this point that the Ultraviolet Purification Systems (UPS) firm loaned their Model EP-50 unit (Figure 1). This unit consists of a 53.6 liter (14.2 gal.) stainless steel cylindrical chamber that houses nine longitudinally-mounted 40-watt UV lamps. Each lamp is enclosed by a quartz sleeve and has an individual ammeter mounted on a control panel. The unit is

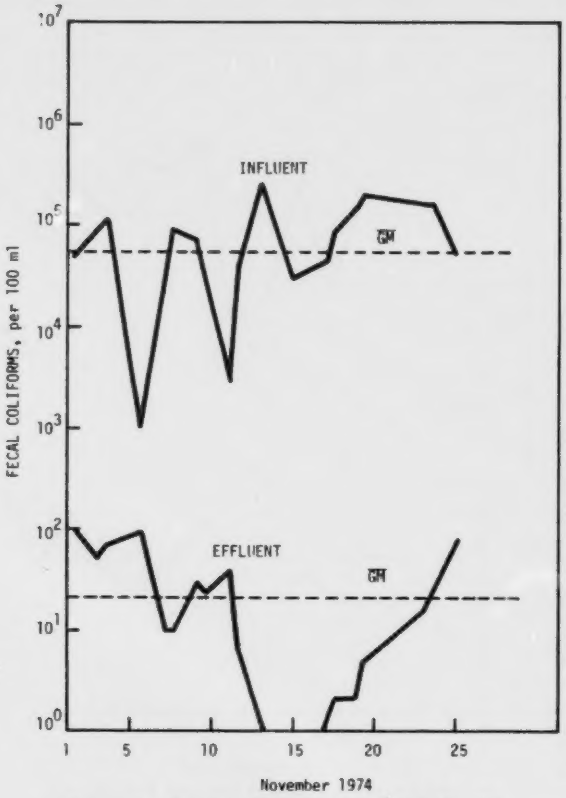

Figure 3. Fecal coliform data for Run No. K4.

equipped with a water quality meter which measures UV light intensity on a unitless scale. Hence, all UV intensity measures in this study were made with an IL 500 radiometer manufactured by International Light, Inc., Newburyport, Massachusetts, through a quartz window on the side of the tank.

The location of the nine UV lamps with respect to the system geometry is shown in Figure 4 which also shows lines of equal intensity calculated for the unit. Flow enters from above at a right angle to the lamps at one end, moves parallel to the lamps toward the other end, and exits at the top at the far end.

The quartz sleeves were cleaned with a solution supplied by the manufacturer. Each run was commenced with a freshly cleaned unit. The cleaning frequency required to keep the system operating at peak efficiency can be expected to vary with the quality of the effluent, but intervals of two to three weeks seem reasonably consistent with the data and temperatures observed.

A total of eight runs were made with the UPS unit varying from 2 days duration to 127 days. All effluent samples of Run Ul of 13 days duration applied to straight secondary effluent at a flow of 1.8 l/sec (29 gpm), resulting in a detention time of 29.8 seconds, met the 200/100 ml fecal coliform criterion (Figure 5). Runs U2 and U3 applied to tertiary settled effluent and to tertiary settled dual-media filtered effluent were even more effective — the latter is shown in Figure 6.

Run U4 was the 2-day run and was short because we only had a 2-day supply of chemicals. Ferric chloride was applied in the Densator (Infilco Co.) prior to UV

Run No.	Type of Effluent Treated	Date Run	Length of Run, days	Composite COD mg/l	Sample NH_3-N mg/l
K1	Secondary effluent	7/26	2½ hrs.	42	1.1
K2	Secondary effluent	9/14-10/7	23	42	3.2
K3	Secondary effluent	10/8-10/30	23	43	1.1
K4	Mixed-media filtered effluent	11/1-11/25	24	14	0.2
K5	Mixed-media filtered effluent	12/7-12/27	20	92	10.1
U1	Secondary effluent	1/23-2/5	13	33	2.2
U2	Tertiary clarified	2/7-2/17	10	35	1.8
U3	Clarified and dual-media filtered	2/19-3/3	12	42	1.5
U4	Chemically clarified	3/6-3/7	2	—	0.2
U5	Secondary effluent	3/8-7/14	127	54	7.1
U6	Dual-media filtered effluent	7/15-8/3	19	107	12.8
U7	Dual-media filtered effluent	8/8-10/5	58	62	4.6
U8	Secondary effluent	10/7-11/30	54	63	5.4
V1	Secondary effluent	4/22	—	46	1.8
V2	Secondary effluent	5/13	—	37	4.4
V3	Secondary effluent	6/26	—	75	10.3

TABLE 1. TYPES OF EFFLUENTS TREATED, TIME AND LENGTH OF RUNS, AND COMPOSITE SAMPLE COD AND NH_3-N MEAN CONCENTRATIONS.

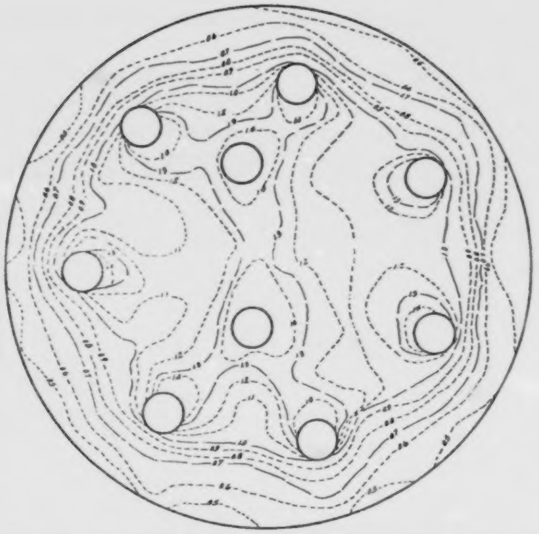

Figure 4. Isointensity patterns for UPS exposure chamber.

irradiation. Geometric mean fecal coliform values of the effluent were less than 4.3/100 ml with a maximum of 33/100 ml.

The shortest UPS run was followed by the longest.

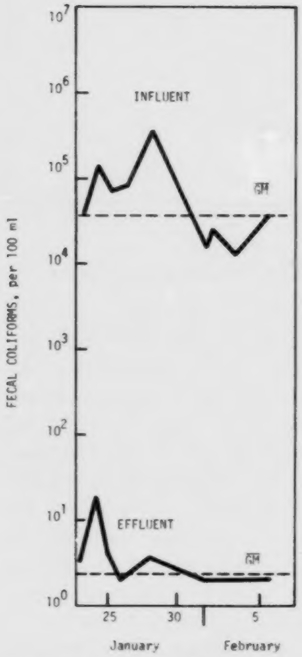

Figure 5. Fecal coliform data for Run No. UI.

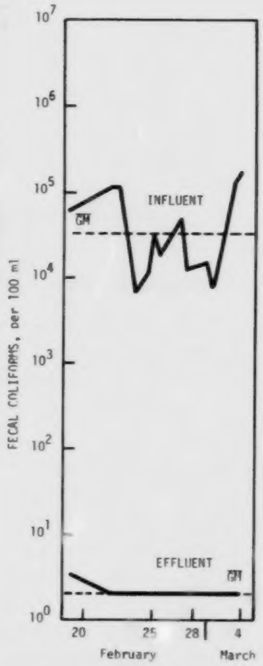

Figure 6. Fecal coliform data for Run No. U3.

Run U5 lasted for 127 days. It was during this run that the oxygen transfer system commenced to fail and many heartaches were encountered in trying to limp along. The fecal coliform limit during this period was not often achieved.

The proprietary oxygen transfer equipment was removed and a home-made diffused air system substituted. It was inadequate to achieve the nitrification desired, but Run U6 using dual-media filtered effluent was nevertheless conducted. The fecal coliform limit was achieved about half the time (Figure 7).

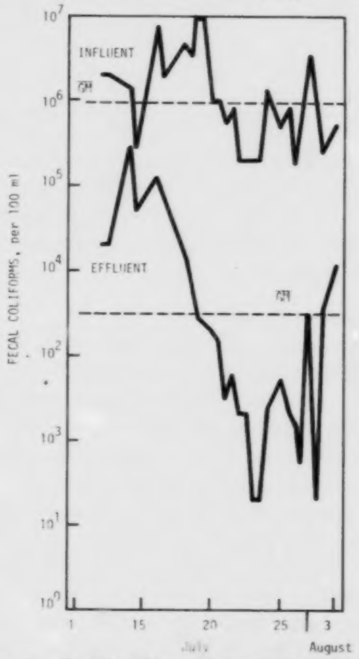

Figure 7. Fecal coliform data for Run No. U6.

In Run U7, new Penberthy oxygen transfer equipment had been installed and the UV system was applied once again to a dual-media filtered effluent. Most of the effluent samples met the fecal coliform limit but, as compared to the earlier runs, the UPS unit during Run U7 was operating near its design hydraulic capacity of 3.1 l/sec (49 gpm) giving a theoretical contact time of 17.3 seconds.

The last run, U8, was made at the maximum flow rate attainable, 3.2 l/sec (51 gpm) which gave a theoretical contact time of 16.8 seconds, and using straight final effluent. Most of the fecal coliform samples exceeded the limit desired during this run (Figure 8). Nitrification was still incomplete — NH_3-N averaging 5.4 mg/l. The resulting effluent COD of 63 mg/l, although higher than when better nitrification is occur-

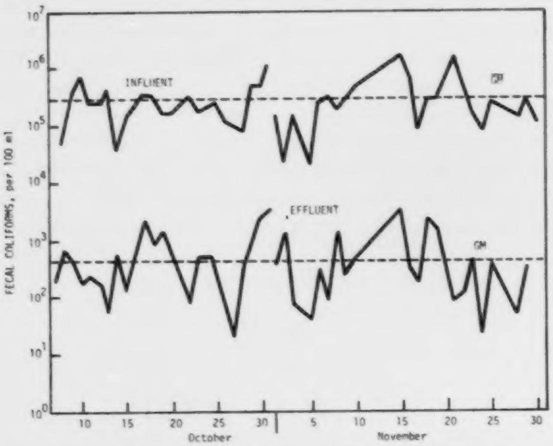

Figure 8. Fecal coliform data for Run No. U8.

ring, is still at a reasonable level for the usual activated sludge effluent. For example, applying the COD: BOD_5 ratio of 2.7:1 results in an effluent BOD_5 of about 23 mg/l.

Virus-Seeding Experiments

The three virus runs were each made at four flow rates which provided exposure periods of 11.4 to 85 seconds (theoretical). The runs were made during the period of equipment difficulties when effluent COD values ranged from 37 to 75 mg/l. The highest COD waters gave the poorest microbiological results, but little difference was observed between phages and poliovirus. The phages ($y = 1.59x - 4.68$, x = log calculated UV dose in μ watt-sec/cm^2, y = log reduction) were possibly a little more resistant than the poliovirus ($y = 1.62x - 4.48$), and both viruses, in turn, were more resistant than the fecal coliforms ($y = 1.48x - 3.21$). Figure 9 shows fecal coliform log reductions as a function of the log of the calculated UV dose. Figure 10 shows the same curve for coliphages. The correlation coefficients for these curves were fairly low (0.70-0.72); hence, additional research is warranted. The no effect dose (determined by setting y = O and solving for x) ranges from 148 (fecal coliforms) to 871 (coliphages) μ watt-sec/cm^2.

CONCLUSIONS

During the course of these experiments, we once again observed that high quality effluents (low COD's) were associated with nitrified operation, a relationship that is even more marked after an adsorption process application (6). Our plots of turbidity vs. transmittance, and suspended solids vs. transmittance yielded shotgun patterns (4). On the other hand, transmittance and COD correlated at 0.76 and transmittance and TOC at 0.95.

Both TOC and COD correlated well with effluent NH_3-N, COD at 0.97 and TOC at 0.88 (Figure 11). The result is the seemingly unlikely correlation of effluent NH_3-N content with transmittance at r = 0.81.

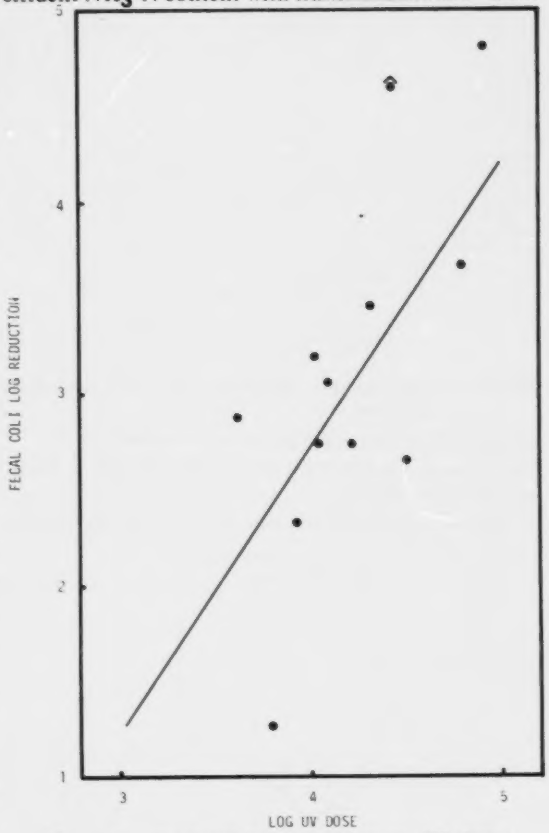

Figure 9. Fecal coliform reduction vs. UV dose

Additionally, highly nitrified operation results in lower coliform concentrations. Figure 12 reflects this observation. The data in Figure 12 compare the average activated sludge effluent NH_3-N concentrations for each K and U run of Table 1 with the mean log fecal coliform content entering the UV unit. The latter value is impacted by the additional unit processes employed such as filtration or tertiary settling. For example, the lowest fecal coliform value on Figure 12 (6.3×10^3) was obtained during Run U4 when chemical clarification was utilized. In spite of these additional treatments, a relationship is apparent. A correlation coefficient was not calculated because two points (↓) are less-than values and one point (↑) is a greater-than value, and also because of the ameliorating effect of the additional treatments on the coliform values.

Light-dark experiments were performed during the course of these studies (6). UV irradiated effluent was directed into two parallel and baffled chlorine-contact basins (without chlorine addition) providing a detention period of about an hour. One basin was covered with black plastic sheet, the other was open. Statistical evaluations of the data showed significant (at a 95% level) regrowth of total coliforms but not of fecal coliforms — although the latter also showed higher effluent concentrations.

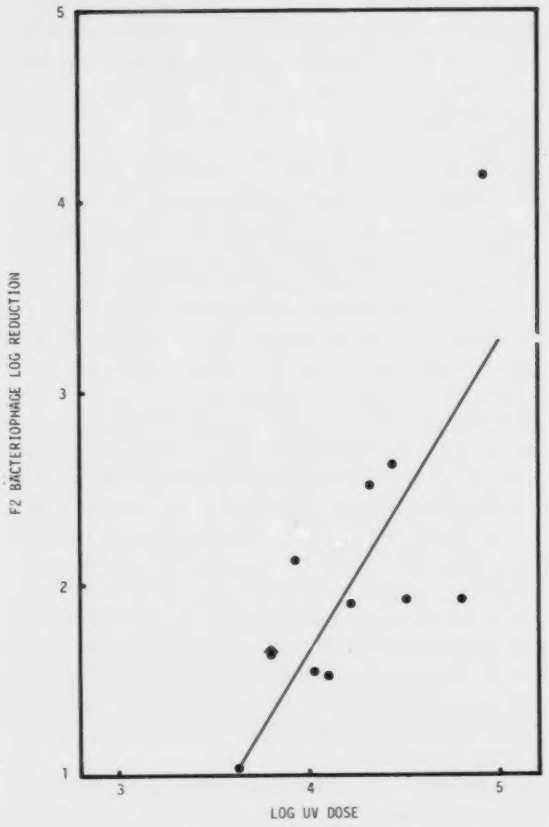

Figure 10. Coliphage reduction vs. UV dose

Clearly, ultraviolet irradiation of secondary effluents is a viable alternative "disinfection" procedure for achieving a 200 fecal coliforms/100 ml effluent criterion. Like ozone, however, UV is quite sensitive to effluent quality variations and will have to be designed to accommodate the expected variations.

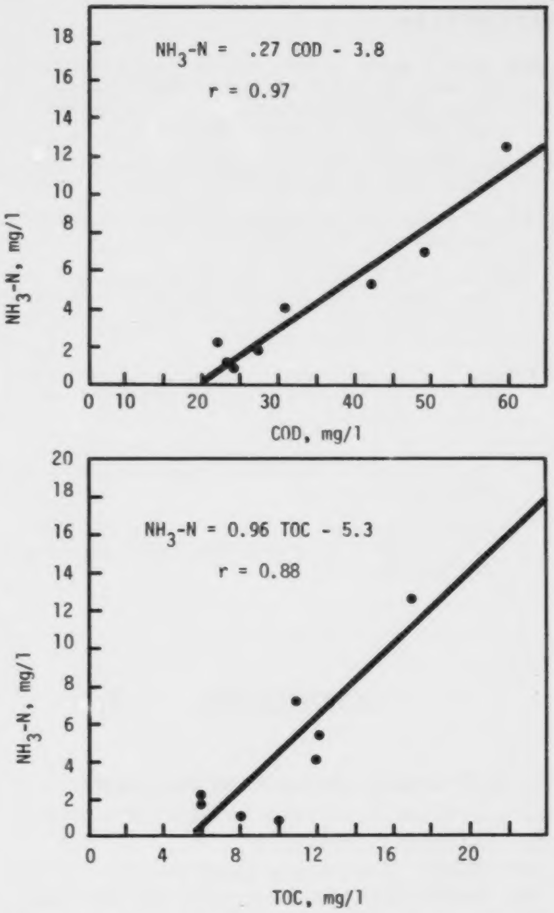

Figure 11. Correlations between ammonia-N and COD and TOC.

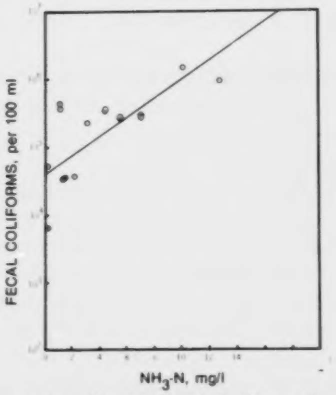

Figure 12. Relationship of Fecal Coliform Concentration to NH_3-N Content of Activated Sludge Effluent

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DISCUSSION

MR. DeSTEFANO, *Riddick and Associates:* Did you do any correlations between UV transmittance and coliform counts?

DR. WOLF: I will have to check that.

MR. DeSTEFANO: It seems to me the correlation between the ammonia and the coliform counts are more correlation between the transmittance and the counts rather than the ammonia itself.

DR. WOLF: I will check that. I have the report with me.

MR. SEVERIN, *FMC Corporation, Chicago:* Did you change your flow rate on this unit during any fecal coliform runs?

DR. WOLF: Yes.

MR. SEVERIN: Did you find a straight line relation with time or did your curves taper off?

DR. WOLF: I had one transparency I was going to show in which we maintained a constant flow rate with the Kelly-Purdy unit and doubled the depth. There was an increased effectiveness of UV due to the increased exposure time which resulted, on the order of about one log. We did vary flow, and flow enters into the UV dose very much by affecting the exposure period. Does that answer your question?

MR. SEVERIN: The reason I am asking is that I have done tests with the EP 50 unit, and found that in primary and filtered secondary effluents there was a linear relationship between survival and $t^{1/3}$ and I am trying to explain this to myself.

DR. WOLF: What kind of factor?

MR. SEVERIN: Time to the one-third.

DR. WOLF: Time to the one-third. I will have to check that out.

MR. SEVERIN: Also, I would like to agree with your suspended solids information. I was able to put all these on a single line, and the indication that you can use a single line for a log reduction with primary effluent as well as with dual media filtered effluent would indicate that suspended solids are not really interfering in the range up to say 100 mg/l.

DR. WOLF: Good. That one has been worrying me right along.

MR. FLUEGGE, *Carborundum Company:* I have tried to go through some calculations in determining what dosages of ultraviolet light are to the wastewater. Looking at your numbers it looks like you have assumed for one thing only one pass of the photon, that the photon is not reflected off the walls or not absorbed by the bulb and then readmitted. Is that what you have done, or did you try to take that into account through measurements?

DR. WOLF: When you get into photons, you are getting out of my field, but we went through two basic types of calculations. This gave us problems all the way through. With the Kelly-Purdy unit we could put the sensing meter on the bottom of the tray, move it along different locations, integrate it and so forth. With the UPS unit we could not get inside to do these things. So we had to do it on a calculated basis, and if you do it by a calculated basis you have some problems because you have a slime buildup on the lamps. It is very slow initially, but once it starts it develops very rapidly, and you also have the problem on the inside of the quartz window.

MR. FLUEGGE: You mean between the bulb and the outer casing.

DR. WOLF: Yes, and so we are relatively insecure with respect to our measurements. When we did the Kelly-Purdy test, for example, we did not even have the UV measuring device at the time, but we went ahead and did it anyway thinking we could come back and measure it, that perhaps there would not be that much deterioration in the bulbs, but when we got into this study it is not as simple as we thought originally. The extinction coefficient becomes terribly important, and so we could not run the extinction coefficient on the early examples any more, so we assumed an extinction coefficient for those particular studies.

MR. FLUEGGE: All of your data then would indicate a constant light source intensity. Is that correct?

DR. WOLF: That is correct.

DR. JOHNSON: Your iso-intensity lines on your diagram there show very low values along the wall. You did consider reflections off the wall.

DR. WOLF: That is a good question. I believe that is correct. I will have to check that out.

QUESTION: Question on maintenance. These units look like they would be maintenance nightmares on a large scale, cleaning them off, especially if you could not easily replace the bulbs or get inside these units. Is that true?

DR. WOLF: There is a lot of proprietary people here and I will let them answer that question, but one of the things that we wanted to do in this study was some cost work with respect to an application in Dallas, and we came up with a pretty large unit for application to Dallas. Furthermore, we just did not have enough information to make a reasonable estimate of cost. So we did not even bother to do it. I also thought we could store the water and get some off-peak power rates and then disinfect during the off-peak power periods, but as I found out the Dallas Light and Power gives power so cheaply to the Dallas Water Utility that they would not even entertain off-peak rates.

QUESTION: Was the smaller unit rated at the same capacity as the large one?

DR. WOLF: It was higher capacity. Almost double.

MR. ELLNER, *Ultraviolet Purification Systems:* Just one point here: the gentleman mentioned the fact earlier that the two units were rated similarly. I think Harold will point out that there is a considerable difference in the UV generators even though you had almost a similar number of lamps in the Kelly-Purdy. There are differences in UV sources, and the ratings of the 30 watt or 40 watt did not refer to UV output but to electrical input, so you cannot make any correlation.

The two quick points that I wanted to make were: there is a tendency to try and describe the acceptability of an effluent based upon visible light determinations, terms such as color and turbidity which are measurements of visible light. I think Harold mentioned earlier in his statement that those factors just do not apply to ultraviolet. You can have situations with very high visible light transmittance but very low UV transmittance, and you can have the opposite under certain circumstances. The point that I would like to make is the combination of all of the factors can be measured with just one measurement, and that is the UV transmittance at 254 nm on a spectrophotometer. That would take into consideration all of these variables, and I was curious, Harold, whether you had some of those ratings such as UV transmission to correlate.

DR. WOLF: Yes, we do, Sid. They are in this report.

MR. ELLNER: And the only other comment at this time is that you will agree that the dosages are theoretical. Would you hang your hat on any of those dosage numbers that appear in the paper?

DR. WOLF: No, I am rather reluctant.

14.

UV DISINFECTION OF SECONDARY EFFLUENT

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ABSTRACT

Ultraviolet disinfection studies are being conducted on a pilot scale using two commercial 230–285l/min (60-75 gpm) units. Potassium ferrioxalate actinometry is used to measure UV dosages. Log reductions of Escherichia coli *in buffered tap water were compared to dosages measured both by the actinometry method and by multiplying the ultraviolet intensity times the retention time. Actinometry dosages were found to have the advantage of being directly related to the average depth of fluid through which the light penetrates and the average intensity within these multiple lamp units. Photoreactivation in UV treated samples exposed to 45 minutes of sunlight caused an average 1.4 log recovery in total coliforms at 25° C. Lower photoreactivation was found at a lower temperature and at a higher UV dose in one of the units. A study of the effects of water quality parameters and unit design on UV disinfection of filtered and unfiltered 2° effluent is still under investigation. Log reductions of total and fecal coliforms are being influenced primarily by the different flow patterns within the units rather than by variations in the parameters or the UV dosages applied. Short-circuiting of fluid through one of the chambers appears to be responsible for this observation.*

INTRODUCTION

Chlorination has to date been the most widely utilized disinfection process because of its low cost, simplicity of operation, and ability to provide residual protection. However, in light of the suspected carcinogenic properties of chlorinated hydrocarbons and the undue stress of chlorine residuals on stream ecology, the use of ultraviolet disinfection is being seriously considered as an alternative. Further justification arises from the fact that chlorination does not reduce wastewater virus concentrations effectively.

Ultraviolet radiation, being a physical agent, is not believed to cause the formation of toxic chemical substances. Also, its germicidal effect is not limited primarily to bacteria. UV absorption by nucleic acids causes disruption of the DNA or RNA molecules which are vital to all organisms including viruses. This suggests that UV radiation should be as effective a disinfectant of the viral component of wastewater as it is of the bacterial component.

In the present study, some aspects of the UV disinfection process which are still poorly understood are under investigation. These are the use of chemical actinometry to measure UV dosages, the occurence of photoreactivation in UV treated samples exposed to sunlight and the effects of water quality parameters on the UV disinfection of filtered and unfiltered secondary effluent.

EXPERIMENTAL SYSTEM

UV disinfection of filtered and unfiltered secondary effluent is being conducted on a pilot scale in Durham, North Carolina at the city's $3.8 \times 10^3 m^3/d$ (1.0 mgd) Sandy Creek contact stabilization plant. The plant produces an effluent of a quality comparable to a typical activated sludge plant. For the unfiltered experiments, water is drawn directly from the plant effluent. For the filtered runs, effluent is diverted into one of three downflow multimedia filters before entering the UV units. Water leaving the units is pumped into a large storage tank where it is subsequently used as backwash water for the filters.

The project employs two commercially available UV sterilization units. Unit No. 1 included an 8 liter cylindrical tank containing 14 25-watt, 38 cm (15 in.) mercury low pressure lamps arranged around the upper two-thirds of the chamber. The lamps are packed closely together to reduce the width of the fluid through which the UV radiation must penetrate. This is referred to as a thin-film design. Water enters the bottom of the unit through a distribution tube and flows up and over a central shaft in somewhat spiral pattern before exiting at the bottom again. Dye studies, shown in Figure 1, indicate that a significant amount of fluid short-circuits the chamber, while other portions are retained for excessively long periods. The former can be seen by noting that the actual retention times are shorter than V/Q, the theoretical retention time.

Unit No. 2 features an 11 liter cylindrical tank housing six 40-watt, 91 cm (36 in.) low pressure mercury lamps. The lamps are not as closely spaced as those in unit No. 1, and thus the reactor operates in a "thick-film" mode. Flow through the chamber is essentially linear with water entering and exiting each end of the unit from the top. Four disk shaped baffles are included to insure adequate mixing. The results of dye studies, shown in Figure 2, indicate a moderate amount of dispersion in the chamber but no short-circuiting or tailing.

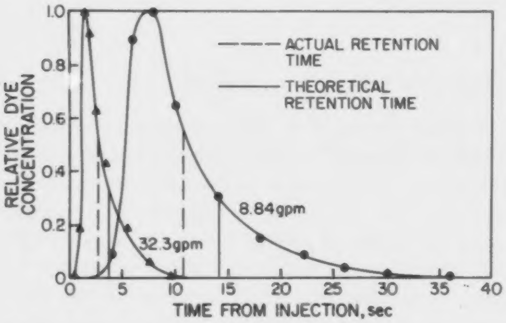

Figure 1. Unit No. 1 dye studies

For both units, the voltage input to the lamps is adjusted via a Variac. Intensity readings are taken at 254 nm with an International Light IL-500 research radiometer calibrated to standards traceable to the National Bureau of Standards. Measurements are made approximately 5 cm (2 in.) above quartz windows situated at the wall of both reactors.

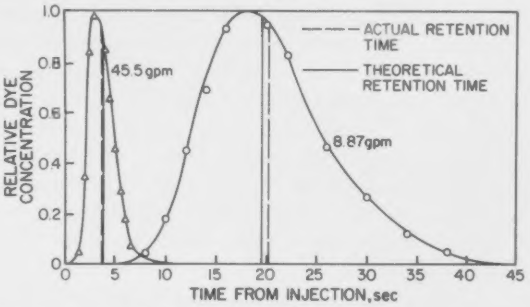

Figure 2. Unit No. 2 dye studies

ACTINOMETRY STUDIES

INTRODUCTION

UV light exhibits a maximum germicidal effect on microorganisms at approximately 265 nm. In most theoretical investigations, the log of the ratio of influent to effluent microbial counts is essentially linear with UV dosage. In wastewater studies, these dose-response curves are usually linear at first, but as dosages increase to more or less saturation levels, the curves begin to tail off before eventually assuming a slope of zero.

A lack of consistency in dose-response data, however, is a problem which is caused by the method used currently to determine UV dosages. Before UV light can be considered a viable alternative to chlorine, a means of determining a dose which more accurately reflects the actual amount of radiation to which the microorganisms are exposed must be found.

Presently, dosages are usually determined by multiplying an intensity measured at the wall of a reactor times the retention time inside the unit. Unfortunately, intensity readings are more dependent upon the particular location of the detector in relation to the lamps than on the actual amount of radiation supplied to the unit. Secondly, the retention time does not describe the actual flow patterns within the unit such as the degree of mixing and the amount of short-circuiting. The geometry of the reactor is a very important variable in UV disinfection, which is almost completely ignored by this method of determining dosages. There is no way in which the relative effectiveness of two different units can be accurately predicted.

There are a number of factors that need to be considered in attempting to obtain a true measure of the average amount of UV radiation that reaches the microorganisms. Ideally, one would prefer a dose which takes into account variables such as the transmission of the fluid, the amount of reflections off the wall, the degree of scattering, the average distance between the lamps, and so forth. Various methods for calculating average intensities have been presented in the literature which deal with some of these factors. These procedures have eliminated the bias due to the position of the UV detector in relation to the lamps and have also taken into consideration the average depth of fluid.

Another way in which a more meaningful UV dose may be obtained is through the use of potassium ferrioxalate actinometry. Solutions containing 0.006N ferrioxalate ion in 0.1 N H_2SO_4 undergo a photochemical decomposition of known quantum yield, Φ, upon exposure to UV light. In the process, ferric iron is reduced to the ferrous form. Because the solution absorbs essentially 100% of the UV emitted by the lamps, and because

$$\Phi = \frac{\#\text{ molecules of product formed}}{\#\text{ photons absorbed}}$$

a measurement of the quantity of ferrous iron produced can be converted to an average number of photons emitted per volume of solution. Thus, dosages are measured in units of energy per unit volume rather than in the customary units of energy per unit area. In the present study, dosages measured in this way and in the conventional fashion were compared to log reduction of *Escherichia coli* in the two units to determine what advantages actinometry might hold over the latter method.

METHODS

The potassium ferrioxalate solutions were prepared in a 1.3 m^3 (350 gal.) polyethylene tank. From there they were pumped through either of the two units at flow rates ranging from 76-285l/min. (20-75 gpm). Samples were collected in brown bottles at three lamp voltage settings for each flow rate tested. In addition, two blanks were collected ahead of the units. To insure that control over the lamps' output was maintained from one experiment to the next, intensity readings were obtained in tap water prior to each run. Intensity measurements could not be made in the actinometry solution itself, as it absorbs all UV light.

Samples were analyzed for ferrous iron spectrophotometrically using 1:10 phenanthroline monohydrate as the colorometric indicator. The exact procedure employed was described by Hatchard and Parker (2). Throughout the analysis, all samples were protected from exposure to light to eliminate any additional photodecomposition of the ferrioxalate ions. The following equation was then used to calculate dosages in μw-sec/cm^3.

$$\text{dose} = \frac{[Fe^{++}]_{sample} - [Fe^{++}]_{blank}}{\Phi \times 1000\ \text{ml/l}} \times \frac{1.196 \times 10^{14}\ \mu\text{W sec}}{254\ \text{nm}}$$

$$\text{where } \Phi = 1.25\ @\ 254\ \text{nm}$$

This calculation is based on the assumptions that 1) the actinometry solution absorbs 100% of the emitted UV radiation and 2) all of the radiation emitted is at 254 nm.

Disinfection studies were carried out by irradiating pure cultures of *E. coli* in buffered tap water over the same range of voltage and flow rates used in the actinometry studies. Organisms were enumerated before and after treatment using the 5 tube multiple-tube fermentation technique as described in *Standard Methods* (1).

RESULTS AND DISCUSSION

Typical results from the actinometry studies are shown in Figures 3 and 4. All curves exhibited some degree of non-linearity, particularly those obtained from unit No. 1. All plots extrapolate to the origin as would be expected except for the dose vs. retention time plots from the first unit. The reason for the exception is probably that actual retention times are significantly shorter than theoretical retention times in that unit.

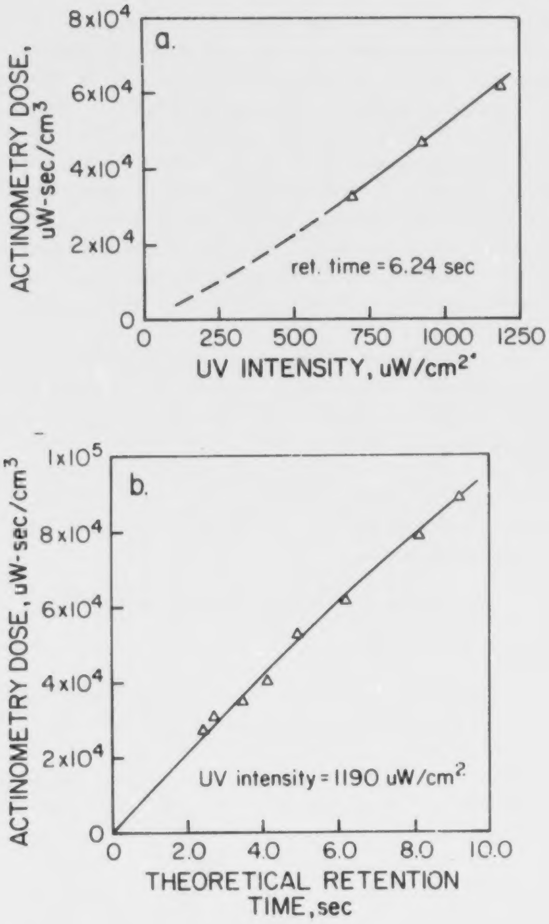

Figure 3. Unit No. 2 actinometry dosages vs. a) UV intensity at reactor wall, and b) retention time

The results of the tap water disinfection studies with pure cultures of *E. coli* are plotted in Figures 5 and 6 vs. the conventional, or intensity times retention time, dose, and the actinometry dose, respectively. Unfortunately, only the tail of the dose-response curve could be obtained because of operational constraints on the system which prevented lower dosage levels from being achieved.

If the actinometry had provided a true measure of the UV dose supplied to the microorganisms, i.e., one that takes into account all of the geometrically related variables such as reflection, short-circuiting and the average depth of fluid, then the two curves in Figure 6 would have been identical. Although they are quite different, there is an even greater lack of similarity between the two curves in Figure 5. Dosages obtained using actinometry appear to control at least some of the factors that are neglected by the conventional dose.

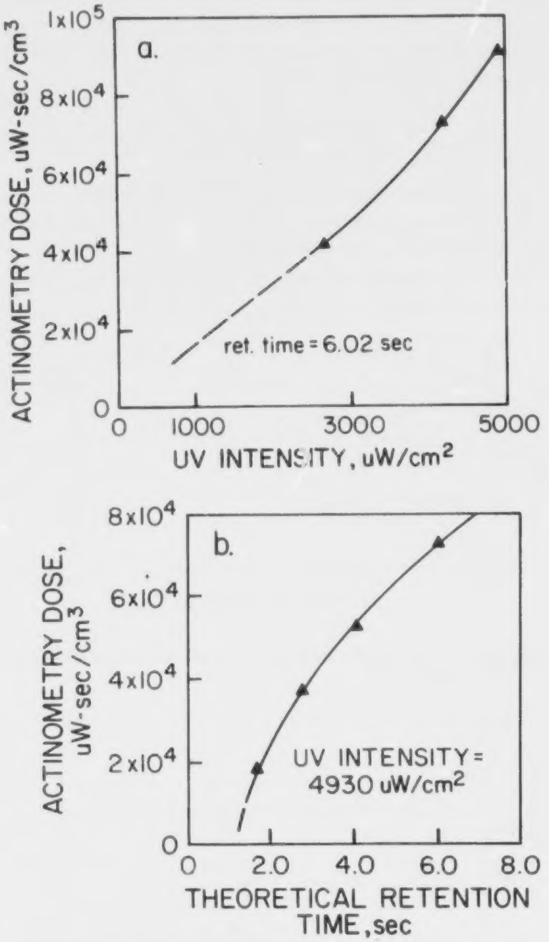

Figure 4. Unit No. 1 actinometry dosages vs. a) UV intensity at reactor wall, and b) retention time

The major difference in the relative position of the curves in Figures 5 and 6 is probably due to the actinometry's elimination of the bias caused by the position of the UV detector. The quartz window through which intensity readings were made was situated much closer to a lamp in unit No. 1 than in unit No. 2. This caused UV intensities and hence conventional dosage measurements on the former unit to be grossly inflated relative to the latter. At a given voltage setting and retention time, the first unit was found to supply a UV dose five times that of the second unit if calculated using I x t. Actinometry dosages, on the other hand, showed only an increase of approximately 1.6 times.

Actinometry dosages provide a measure of the total number of photons emitted by the lamps per unit volume and time. This dose is directly related to the average depth of fluid through which the light passes if the latter is defined as the

$$\frac{\text{total volume of unit}}{\text{surface area of lamps}}$$

This is because the total number of photons a lamp emits is proportional to its surface area. The average depth of fluid is an important treatment parameter which actinometry dosage measurements inherently take into account.

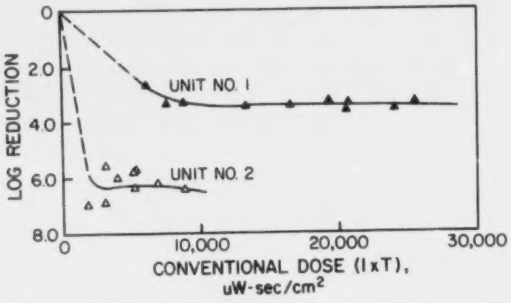

Figure 5. Log reductions of E. Coli vs. conventional dose

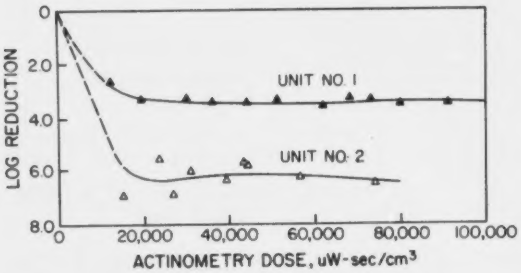

Figure 6. Log reductions of E. Coli vs. actinometry dose

The differences between the two curves in Figure 6 emphasize the failure of the actinometry method to control all of the important variables mentioned earlier; in particular, the degree of short-circuiting in the two units. For example, the first unit was unable to achieve log reductions greater than 3.6 probably because a small percentage of the fluid short-circuited the chamber. This short-circuiting, however, would not be expected to have as significant an effect on actinometry dosages as it would on disinfection efficiencies, which are based on a logarithmic scale.

Actinometry would also not be expected to control variables such as the degree of reflection of UV light off the reactor walls and the configuration of the lamps in relation to the overall unit. This is because the solution absorbs almost 100% of the radiation within a depth of only 3-4 millimeters. Finally, the effects of UV absorbing substances present in the waters to be treated are not considered. For this reason, attempts to correlate actinometry dosages with log reductions of microorganisms in wastewaters of widely varying characteristics are meaningless.

UV dosages measured using chemical actinometry are more useful than those obtained by the conventional method because they are related directly to the average depth of fluid through which the light passes. Although not all of the treatment variables involved are controlled by this method, it is still a potentially useful technique for comparing and calibrating UV disinfection units.

PHOTOREACTIVATION STUDIES

INTRODUCTION

An additional problem may be associated with the use of UV light to disinfect wastewater effluents. A situation may arise where organisms supposedly inactivated by UV radiation may restore their reproductive abilities upon exposure to visible light. Such an event is termed photoreactivation (PR). It has been defined by Jagger (3) as "the restoration of ultraviolet lesions in a biological system with light of wavelength longer than that of the damaging radiation". The repair mechanism of PR is activated upon exposure to light of wavelength between 300 and 500 nm; a major portion is stimulated in the region of 355-385 nm for most microorganisms. A phenomenon such as photoreactivation must be taken into consideration when evaluating the overall efficiency of an ultraviolet disinfection process.

METHODS

Samples of secondary effluent treated and untreated by both units were collected in sterile bottles wrapped in aluminum foil. Flow rates and voltage levels used were comparable to those employed in the actinometry studies. After being placed immediately on ice, the samples were transported back to the laboratory while being kept completely unexposed to light. Sample aliquots were pipetted into petri dishes labelled as follows: 1) influent dark control, 2) effluent dark control (wrapped in aluminum foil), 3) influent sunlight exposed and 4) effluent sunlight exposed. All dishes, control and exposed, were kept in a constant temperature bath for forty-five minutes in direct sunlight. The intensity of PR light was measured at 15 minute invervals with a General Electric light meter equipped with a glass 320-390 nm filter. After completing the exposure, the samples were immediately placed in a dark, 4°C refrigerator until further use within the hour. Total coliforms were enumerated using the membrane filter technique as recommended in *Standard Methods* (1).

RESULTS AND DISCUSSION

The initial point of interest concerning the photoreactivation of organisms in a wastewater environment dealt with the effects of temperature. Influent and effluent samples collected from unit No. 2 were exposed to sunlight as described earlier. Temperature baths at 4°C and 25°C were used to compare PR simultaneously. As shown in Table 1, the disinfection in the dark controls at each temperature was consistantly higher than in their counterparts exposed to sunlight. The decrease in coliform kills may be attributed to the organisms ability to photoreactivate and resume replication. Also of interest is the fact that sunlight exposed log reductions at 25°C are approximately 40% less than those at 4°C. This decrease, although small, is consistant and suggests that PR is more probable in a 25°C environment than at 4°C.

TABLE 1. EFFECTS OF TEMPERATURE ON PHOTOREACTIVATION

		Dark Control	Sunlight Exposure
4°C	Mean Log Reduction	2.07	1.59
	Std. Deviation	0.14	0.58
	95% Confidence Interval	1.73-2.41	0.61-3.02
25°C	Mean Log Reduction	2.07	0.93
	Std. Deviation	0.14	0.21
	95% C.I.	1.94-2.20	0.40-1.46

An interesting result which arose during the temperature study was the decrease in influent total coliform counts from the dark control to the sunlight exposed sample. At both 4° and 25°C, a small but consistant decrease was observed between all dark and light samples, especially with the first run, where greater than one log kills were found. UV light disinfects coliforms most effectively in the region of 265 nm, yet very little energy of wavelengths shorter than 290 nm are found at the earth's surface. The results thus suggest that disinfection due to the action of longer wavelengths is occuring simultaneously with photoreactivation. During sunlight exposure, these two phenomena compete with one another, with photoreactivation usually predominating.

Samples from unit No. 2 exposed to high UV dosages behaved differently upon exposure to sunlight from those exposed to a lower dose. A finding consistant with previous data was the second unit's superior performance relative to unit No. 1 in the dark controls. The result was further manifested at high dosages in the relative degree of photoreactivation observed in samples treated by the two units. The relatively ineffective unit No. 1 inflicted only minor damage to most of the inactivated organisms, thus allowing them to make use of the PR light. See Table 2. The second unit, on the other hand, operated with such a high efficiency that no organisms were capable of repairing themselves. In fact, additional disinfection was observed when these samples were exposed to sunlight.

TABLE 2. PHOTOREACTIVATION USING HIGH UV DOSE

		Dark Control	Sunlight Exposure
Unit No. 1	Mean Log Reduction	3.24	2.08
	Std. Deviation	0.14	0.45
	95% C.I.	3.14-3.38	1.60-2.55
Unit No. 2	Mean Log Reduction	4.01	5.13
	Std. Deviation	0.08	0.54
	95% C.I.	3.93-4.09	4.56-5.70

This study's approach to photoreactivation was unusual in that PR has always been studied with pure cultures of microorganisms rather than the heterogeneous populations characteristic of municipal effluents. As such, control over variables such as the chemical and biological characteristics of the wastewater and the intensity of sunlight could not be exercised in these experiments. Nevertheless, the results show quite clearly that the occurence of photoreactivation may significantly reduce the efficiency of wastewater UV disinfection.

WASTEWATER STUDIES

INTRODUCTION

A pilot plant study is being conducted to define the water quality parameters useful for predicting and monitoring the effectiveness of a UV treatment operation in wastewater. Many parameters are therefore being measured in this study in an attempt to isolate those most directly related to disinfection efficiencies. Two independent variables are also being investigated: 1) filtered vs. unfiltered effluent and 2) lamp spacing in the UV units. The thin-film design characteristic of the first unit has potential advantages over the thick-film arrangement particularly in water with a high UV absorbance.

METHODS

The experimental design includes four variables, each of which can assume two conditions: 1) UV unit (No. 1 or No. 2), 2) flow rate (approximately 130 or 285 l/min), 3) voltage input to lamps (60 V or 127 V), and 4) quality of the influent to the units (filtered or unfiltered). Batch experiments are performed on each unit using randomly determined combinations of the six possible conditions. A total of nine repeats of each set of conditions will eventually be performed.

Samples used for chemical analysis are collected in 3.8 liter plastic containers, while those for bacteriological examination are obtained using sterile brown glass bottles. Lamps are allowed to warm up to 35-40°C before the samples are taken.

In an effort to correlate parameters of water quality with the efficiencies of the UV units, a battery of chemical tests are performed on all samples using procedures outlined in *Standard Methods* (1). Influent wastewater to the units is analyzed for the following:

1. Temperature
2. UV absorbance (Beckman DB spectrophotometer)
3. Alkalinity
4. Conductivity (Radiometer model CDM 2e)
5. Suspended solids
6. Nitrogen forms (NH_3, NO_2^-, NO_3^-, Auto Analyzer)
7. Total Iron (Atomic Absorption Spectrophotometer)
8. Total Organic Carbon (Beckman combustion IR carbon analyzer)
9. Turbidity (Hach 2424 nephelometer)
10. pH
11. Chemical Oxygen Demand

Effluent samples from the units are examined for temperature, dissolved oxygen (YSI Model 51B), nitrogen forms, conductivity, TOC, COD, and pH.

The 5 tube multiple-tube fermentation technique as outlined in *Standard Methods* (1) is employed for the enumeration of total and fecal coliforms in all samples. Results are reported as log reductions in coliform counts. The procedure is performed to the confirmed level with lauryl tryptose utilized as the presumptive medium.

RESULTS AND DISCUSSION

Approximately 30% of the experiments have been completed thus far. The data have been correlated and preliminary efforts to analyze the results for significant trends have been made.

Two findings are particularly noteworthy at this time: 1) unit No. 2 performs much more efficiently than unit No. 1 under all conditions, and 2) flow rate and lamp voltage have almost no effect on log reductions of coliforms at the levels tested. Both of these observations are consistant with the pure culture tap water results discussed earlier. Apparently, at even the lowest possible UV dosage level available, the units are operating where there is little increase in inactivation with increased dose. In this region, the major source of variation in the coliform log reductions is the differences between the units themselves. The second unit gave a mean log reduction of 3.56 in total coliform while the first unit gave only 1.92 logs inactivation.

In this investigation, conclusions regarding the advantages of the thin-film vs. thick-film concept in reactor design will not be possible. As shown in Figures 1 and 2, differences in the flow patterns within the units are obscuring any possible effect of film thickness. Although unit No. 1, with its 14 closely spaced lamps, yields a UV actinometry dose 1.6 times unit No. 2, short-circuiting in the reactor appears to be preventing it from achieving high log reductions of coliforms.

In attempting to correlate the various water quality parameters and process control variables with UV disinfection, problems have been encountered so far due to the overpowering effect of the unit on log reductions. For example, a strong *negative* correlation between log reductions and UV intensity was obtained simply because the less effective unit No. 1 actually yields a higher UV intensity reading than does the second unit. Plots of UV intensity vs. log reductions for each unit separately show no correlation. Most of the water quality parameters exhibit little or no correlation with log reductions at this time. However, once the statistical analysis is performed with respect to each unit, the importance of these variables will become clearer.

SUMMARY AND CONCLUSIONS

Ultraviolet disinfection studies are being conducted at a pilot plant using two commercially available 230 to 285 l/min (60-75 gpm) UV units. The use of potassium ferrioxalate actinometry to measure UV dosages has been found to have advantages over the conventional method. Actinometry dosages are directly related to the average intensity of all lamps and the average depth of fluid in the units, while conventional dosages are biased by the position of the UV detector in relation to the lamps.

Results from photoreactivation studies of UV inactivated coliform organisms exposed to sunlight show a 1 to 2 log recovery in total coliforms at 25°C. Lower recoveries were found at a lower temperature. Under high dosage conditions, photoreactivation was found to be a function of the unit, with samples from unit No. 2 actually showing additional disinfection upon exposure to sunlight.

Ultraviolet disinfection studies on filtered and unfiltered secondary effluent are still in progress. Only small effects of water quality parameters and UV dosages on log reductions of total and fecal coliforms have been found so far. The primary variable affecting inactivation is the non-ideal flow patterns of the two units. This result is due probably to the effect that short-circuiting is having on the effectiveness of unit No. 1. The importance of the other variables will become clearer following a more complete analysis of the data.

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2. Hatchard, C. G. and Č. A. Parker. 1956. "A New Sensitive Chemical Actinometer II. Potassium Ferrioxalate as a Standard Chemical Actinometer." *Proc. Royal Soc. London*. A235:518.
3. Jagger, J. 1958. "Photoreactivation". *Bac. Rev.* 22:99.

DISCUSSION

MR. HEINSOHN, *Portstar Ind., Inc.*: Clarify for me unit 1 and 2. Which was the thin film?

MR. ALDRICH: The thin film was unit number 1, and the thick film is unit number 2.

MR. HEINSOHN: Unit number 2 was internally baffled. Is that correct?

MR. ALDRICH: Yes. It has four internal baffles.

MR. TONELLI, *Ontario Ministry of the Environment*: Apart from being a descriptive term, what does the term "thin film" versus "thick film" actually mean as regards to fundamentals of design?

MR. ALDRICH: It refers to the lamp spacing within the units. I am sure there is an arbitrary cutoff where you define something as thick film and where you define it as thin film so it is more of a qualitive kind of comparison. It relates to how closely spaced the lamps are together, and particularly what the average depth of fluid is through which the light must penetrate.

MR. TONELLI: From your actinometry results, you do not feel there is anything fundamentally different. Is it just a matter of definition or semantics?

MR. ALDRICH: Are you talking about differences between measuring the average depth and the actinometry doses?

MR. TONELLI: No, I am still talking about the thin film and thick film concept, because on the face of it there seems to be an infinity of intermediate thin films and micro-thin films. Was there any theoretical basis for selection of depth or separation of lamps for the study, or were there two units that were available?

MR. ALDRICH: No. We originally were interested in comparing the two units on the basis of the thin film versus the thick film. That was one of the original objectives of the study. As far as the actinometry is concerned, the spacing of the lamps did not really have that much to do with the actinometry. The spacing is related to the average depth of fluid and so is the actinometry, but other than that there really was no particular basis for comparison.

MR. DeSTEFANO: I have a question about the photoreactivation study. Did you do any counts immediately after collecting from the effluent, because there is also a phenomenon of dark repair and I would think that your control says that photoreactivation is a more powerful repair mechanism than dark repair is.

MISS ELLIOTT: I did not do any immediately afterwards. There was a matter of transport time in getting everything set up, and before we exposed these samples they were kept in a dark refrigerator at about 4°C. So I did not do any immediate ones afterwards.

MR. DeSTEFANO: But dark repair could have occurred in forty-five minutes?

MISS ELLIOTT: There was a possibility, but I would think that the cool environment of the refrigerator would possibly prevent that.

MR. VENOSA: I would like to make one particular comment. Our original idea in this project was that we wanted to compare thin film versus thick film, and there is no clear distinction between thin film and thick film. There is one company that makes a thin film type design, and this refers to approximately 0.25 inch of water wall. That is the distance the light traverses through the water. The distance between any two quartz sleeves is approximately 0.5 inch. Therefore, the water wall is 0.25 inch. The other unit is approximately 1 to 1.5 inches. The concept is what we were interested in, i.e., thin film, high intensity, short detention time, versus thicker water wall, long detention time, lower intensity.

MR. TRAVER, *EPA*: Any type indication we have had as far as UV disinfection deals with cleansing of the ultraviolet lamp sources in a submerged condition, be it either some type of a cleansing solution or mechanical light system. I do not remember or recall in the presentation any indication of this situation. Did you have to go on a down time basis, or how was this attempted in units 1 and 2?

MR. ALDRICH: We are not running any continuous experiments at this time. All our experiments are short term, and the two units are different with respect to the cleaning mechanism. Unit number 1 has a mechanical cleaning system powered by compressed air, and unit number 2 is cleaned with the use of a cleaning solution which is circulated through the chamber. We have not run into any problems with down time since we are not running continuously, and both of the systems seem to work quite well at this time.

MR. TRAVER: Can unit number 1, using a mechanical wiper system, be cleaned while in operation, or does it have to go down?

MR. ALDRICH: No, it is cleaned while operating.

15.

FULL SCALE EVALUATION OF ULTRAVIOLET DISINFECTION OF A SECONDARY EFFLUENT

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INTRODUCTION

This presentation is a progress report on a full scale evaluation of U.V. Disinfection of an effluent from a secondary wastewater treatment plant. The study is funded by the U.S.E.P.A. (MERL) and the Northwest Bergen County Sewer Authority. The principal investigator for the study is Hydroscience, Inc., Westwood, New Jersey. The ultraviolet equipment was developed and is manufactured by Pure Water Systems, who operate and maintain the on-site equipment.

The tasks involved in the overall program included the installation and shakedown of the U.V. equipment. Experimental work was then directed to an evaluation of the system under various operational conditions to determine dosage requirements relative to disinfection efficiency. Earlier in the program the system was continuously monitored during a viral sampling program conducted by the Carborundum Company, the results of which are to be reported in a separate presentation. The phenomenon of photoreactivation was evaluated concurrently with the primary sampling program, and the system was continually monitored for operation and maintenance efficiency. The final report to the EPA is expected in the spring of 1979.

SITE LOCATION

The site of the installation is the Northwest Bergen County Water Pollution Control Plant, located in Waldwick, New Jersey. Figure 1 presents a site schematic of the plant. It is a conventional air activated sludge plant with a design capacity of 30,000 m^3/day (8 mgd), and an average yearly flow, at present, of approximately 18,900 m^3/day (5 mgd). Completed in 1968, the plant is a modern, efficient facility discharging a well treated secondary effluent to Ho-Ho-Kus Brook, a water quality stream.

The plant has dual chlorine contact chambers. Under present flow conditions, one has remained inactive. This provided an ideal location for the installation of the gravity feed U.V. disinfection system.

EQUIPMENT INSTALLATION AND SPECIFICATIONS

A plan view of the chlorine contact chambers presented in Figure 2 shows the overall U.V. installation. To the left is the active chlorination system, and on the right the inactive chlorine contact chamber which provided the site location. The unit itself was installed at the head end of the chamber, the influent flow rate being controlled by weir gates on the influent channel. The U.V. unit was set into the channel and a platform set above it to support the ballast and provide a work area. Flow was measured by 4 V-notch weirs located at the effluent end of the contact chamber with a level sensor and recorder. A Cl_2 diffuser was located approximately 6 to 9 meters downstream of the U.V. unit in compliance with New Jersey law.

A cutout view of the Cl_2 contact chamber on Figure 3 shows the actual installation of the U.V. lamp battery. Two concrete webs provide the support. A pump is placed between the webs to keep the space dry. The lamp battery itself is supported by two steel bulkheads set into the concrete webs, with an air seal similar to a rubber inner tube along the perimeter between the bulkhead and concrete webs. Figure 3 also shows the mechanical wiper and the pneumatic cylinder which drives the wiper mechanism.

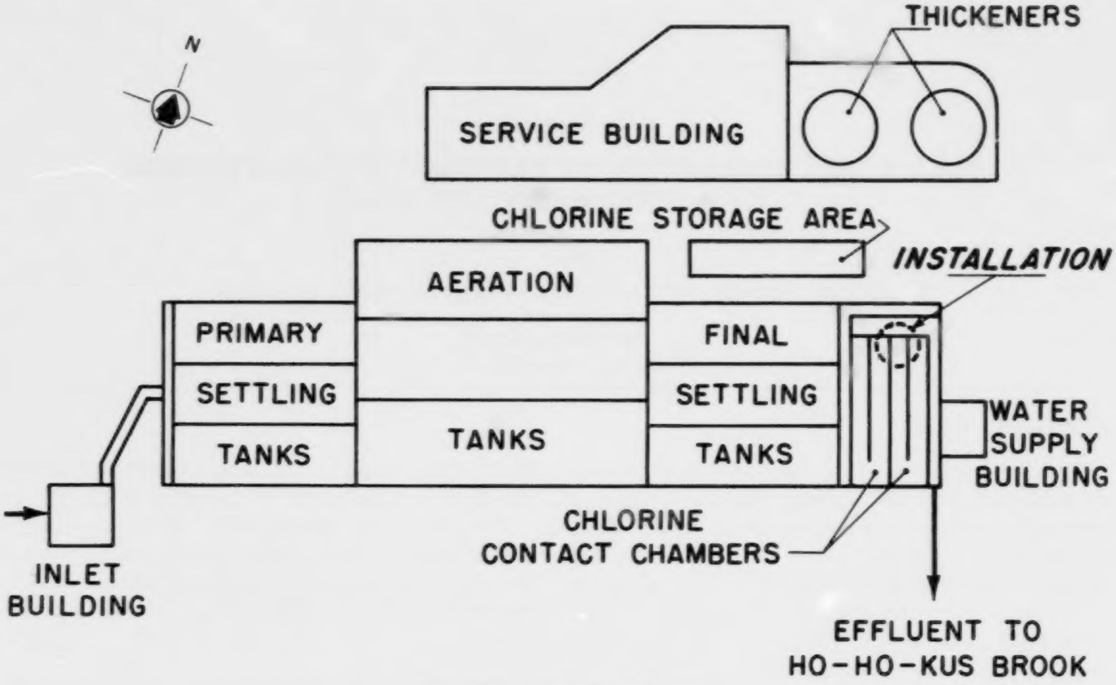

FIGURE 1. SITE SCHEMATIC

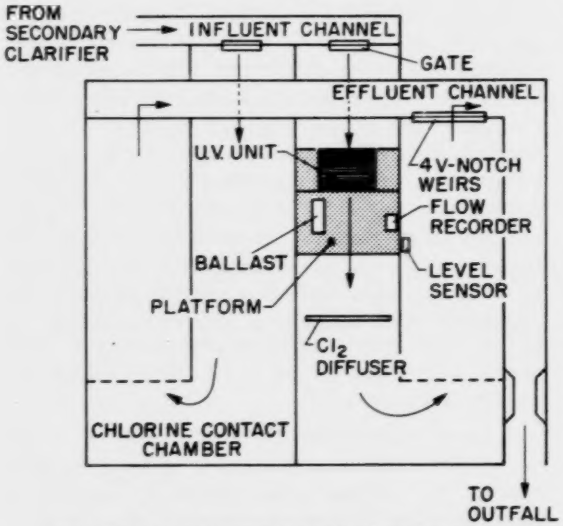

FIGURE 2. PLAN VIEW OF ULTRAVIOLET UNIT INSTALLATION

The specifications of the unit used at Northwest Bergen County are presented in Figure 4. It has a total of 400 germicidal lamps. They are 142 cm (6 ft long) 85 W lamps with an output of 30 W in the germicidal U.V. range. Each lamp is jacketed in quartz. The total effective arc length of the unit is 610 meters (2,000 ft).

Total power consumption by the unit is 45 KVA at an operating voltage of 480 V. The unit is capable of lamp battery shutoff in 1/6 increments and power variability from 40 to 100% full power.

The overall dimensions of the unit are 76 x 76 x 142 cm (3 x 3 x 6 ft) with a void volume of 0.63m^3 (22.2 ft^3). Head loss is estimated at 15 cm (6 in) at a flow rate of 21,000 m^3/day (5.5 mgd). The mechanical wiper mechanism is comprised of replaceable elastomeric glands fitted over each of the quartz tubes. The wipers are cable driven at a variable stroke rate by a pneumatic cylinder. A unique feature of the U.V. unit it the utilization of the "thin film" concept, which is induced by the spacing of the lamps. The nominal liquid film thickness is 0.6 cm (0.25 in).

EXPERIMENTAL PROGRAM

The data reported herein represent the analysis period of June through August, 1978. During this time, two randomized sampling series were conducted. The first, conducted in June and July, evaluated the system at two specific flow rates and four applied voltage settings. The second series evaluated the

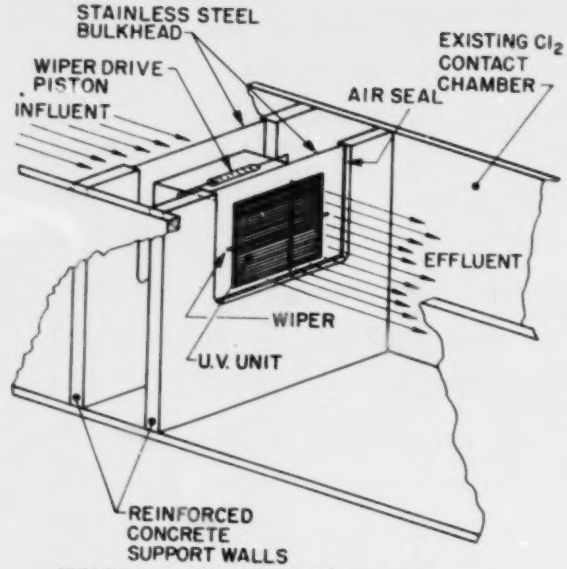

FIGURE 3. ULTRAVIOLET DISINFECTION UNIT INSTALLATION

SPECIFICATIONS

MODEL : PWS SE-7.5
LAMPS : NUMBER 400
TYPE : 85 W PWS-L60; 152 CM LENGTH
30 W AT 2537 Å
2.3 CM. Ø QUARTZ JACKETING
TOTAL EFFECTIVE ARC LENGTH : 610 METERS
TOTAL POWER : 45 KVA
OPERATING VOLTAGE : 480 VAC/60 HZ/3 PH
LAMP BATTERY SHUTOFF IN 1/6 INCREMENTS
POWER VARIABILITY : 40 TO 100 %
DIMENSIONS : 76 x 76 x 152 CM.
VOID VOLUME : 0.63 M^3
HEAD LOSS ~ 15 CM. AT 21,000 M^3/DAY
WIPER : REPLACEABLE ELASTOMERIC GLANDS, CABLE DRIVE OFF PNEUMATIC CYLINDER

FIGURE 4. U.V. DISINFECTION UNIT

JUNE, JULY, AUGUST, 1978
(119 SAMPLES)

	MEAN	MIN.	MAX.
TOTAL COLIFORM	3.63×10^5	3.2×10^4	1.6×10^7
FECAL COLIFORM	9.5×10^4	1×10^4	3.2×10^6
COD (F)	26.0	14.0	47.0
SS	6.0	1.0	18.0
TURBIDITY	4.0	1.0	25.0
pH		6.5	7.6
TKN (F)	11.0	3.0	24.0
NO_2-N	1.1	0.3	3.7
NO_3-N	8.9	1.8	20.0
NH_3-N	9.0	1.9	20.0
COLOR	34.0	18.0	75.0
U.V. ABSORBANCE (T)(1)	0.171	0.277	0.137
% TRANSMITTANCE(1)	67.4	52.8	72.9

(1) 1 CM. CELL AT 253.7 nm

FIGURE 5. SUMMARY · INFLUENT ANALYSIS

system at a single flow rate with variable lamp operation and variable applied voltage settings.

Sampling was conducted two days per week at eight operational modes per day. Thus, sixteen samplings were conducted per week. The sequence of operational conditions was randomly selected to minimize bias due to variations in water quality. The randomized sampling series conducted during the June through August period were part of an intensive sampling program, statistically designed to provide an evaluation of the system relative to bactericidal efficiency under a wide range of ultraviolet energy dosage applications.

RESULTS

Wastewater Characterization

Figure 5 is a summary of the analyses performed on the influent to the U.V. unit. Sampling was by grab only, in sterile opaque glass bottles. Total and fecal coliform densities were measured by the 5-tube multiple tube MPN procedure. All influent grabs were analyzed for the specific wastewater quality parameters shown on Figure 5. The mean densities of total and fecal coliform were 3.6×10^5 and 9.5×10^4, respectively. The influent wastewater was relatively stable, and highly indicative of quality treatment; the COD averaged 26 mg/l, while the turbidity and SS were relatively low at 4 FTU and 6 mg/l, respectively. The water temperature averaged 22° C. As shown by the distribution of the nitrogen species, the plant was operating in a nitrification mode during the three month period. U.V. transmittance was measured in 1 cm quartz cells. The average transmittance was 67%, with a range of 52 to 73%. This transmittance level falls in the range indicative of a high quality secondary effluent.

Approximately one half of the effluent samples collected were analyzed for these same parameters, the results of which were tested against the influent analyses to determine if there was any change upon irradiation. A Student's t analysis at the 95% confidence level indicated no significant alteration in the wastewater parameters measured. The results of this analysis are summarized on Figure 6.

Ho: MEAN Δ = 0.0

INF. TO EFF.

(56 SAMPLES)

	MEAN Δ	CALC. t (1)
COD (T)	−0.48	0.87
COD (F)	−1.0	1.71
COLOR	−0.8	0.9
U.V. ABSORBANCE (T)	−0.0028	1.39
U.V. ABSORBANCE (F)	0.0007	0.53
TSS	0.19	1.37
TURBIDITY	0.18	0.84
TKN (F)	−2.45	1.37
NO_2 −N	0.011	0.73
NO_3 −N	0.041	0.55
NH_3 −N	−0.085	0.79

(1) SIGNIFICANT AT t >2.005

FIGURE 6. TEST OF DIFFERENCES

Disinfection Efficiency

Before presenting the disinfection results, the manner in which dosage is reported should be explained. It is computed simply as the applied germicidal power (KW) divided by the flow rate (m^3/sec):

D = AGP/Q
where D = dosage (KW-sec/m^3)
AGP = applied germicidal power (KW)
Q = flow (m^3/sec)

This dosage unit, presented for use in conjunction with, or as an alternate to the standard μw-sec/cm^2 term, is reportable in both total and germicidal power applications, and may present a practical procedure for sizing and comparing alternative U.V. equipment. The reader is cautioned, however, that this type of dosage unit used alone represents a "black box" approach. Consideration must be given to intensity levels and water quality. It is anticipated that this study will provide a greater input to a more effective U.V. sizing parameter.

Figure 7 presents a composite of all coliform data collected during the three month period, representing a total of 119 samplings. The data are presented as a log-log relationship of surviving fraction to dosage. Least squares analysis was performed to compute the regression lines as shown for both TC and FC. As the figures indicate, the range of dosage levels investigated was approximately 4 to 90 KW-sec/m^3, representing exposure times in the order of 0.3 to 5 seconds. In the situation of the Northwest Bergen plant, a 99.9% removal would require a dosage of 35 KW-sec/m^3 under average flow conditions and an exposure time of approximately 2-2.5 seconds.

Design Nomograph

Assuming a linear relationship between log surviving fraction and a log dosage (Figure 7), a design nomograph was developed relating influent flow and expected influent coliform densities to germicidal power requirements. The nomograph presented in Figure 8 is based on a desired effluent average fecal coliform density of <200 MPN/100 ml.

Equipment sizing would be based on peak flow, which is assumed to be twice the average design flow of the plant. As an example, if a plant is to be designed at an average flow capacity of 30,000 m^3/day (8 mgd), the peak flow condition would be 60,000 m^3/day (16 mgd). If the expected influent fecal coliform density is 10^5 MPN/100 ml, Figure 8 indicates an estimated germicidal power requirement of 18 KW. Utilizing lamps with a germicidal output of 30 W/lamp, the implied lamp requirement would be 600. Similarly,

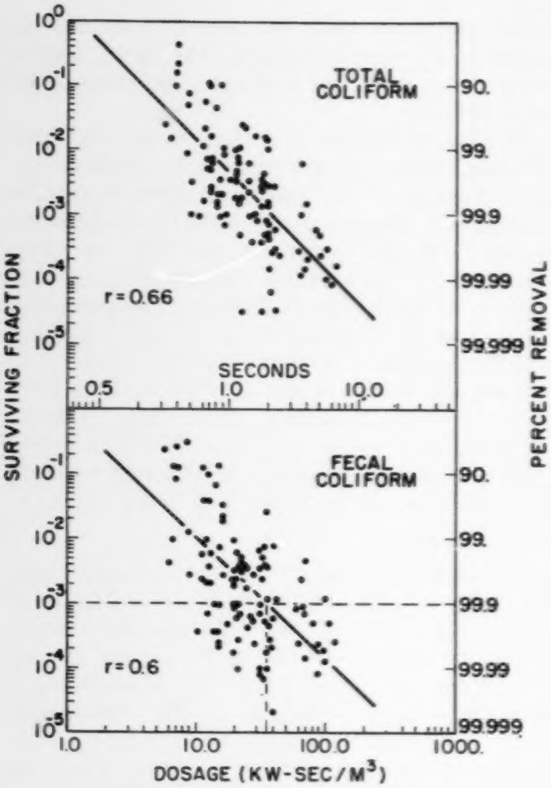

FIGURE 7. SURVIVING FRACTION

assuming a total power consumption of 110 W per lamp, the total power application becomes 66 KW.

As indicated by Figure 8, the assumption of a linear relationship in log surviving fraction with log dosage induces a sensitivity of the system design to influent coliform densities. Single log increments in influent density levels will affect system design requirements by a factor between 3 and 3.5.

Preliminary Cost Estimates

Costs were developed on a preliminary basis using equipment purchase figures provided by the manufacturer and the design nomograph shown on Figure 8. The costs included in the capital cost estimates are the equipment purchase (frame, unit and control panel, and installation), and excludes the support structure and any ancillary equipment requirements. Design and cost estimates for these have not been developed as yet. Design of the U.V. equipment is based on peak flow conditions and amortization is over a twenty year period at an interest rate of 6⅝%.

Operation and maintenance costs are reported on an average flow basis. Service is estimated at 15% of annualized capital costs. It is presently assumed that the lamps would need to be replaced at an annual rate. Power costs were assumed to be 3.5¢/KWH.

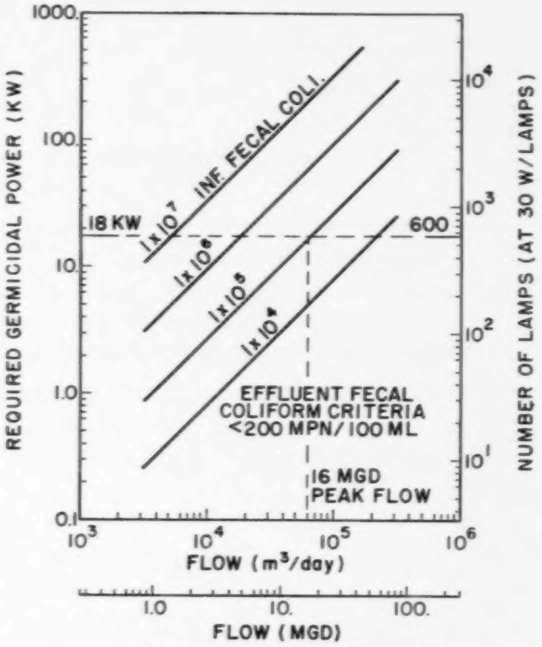

FIGURE 8. DESIGN NOMOGRAPH

In Figure 9, costs were related to the design peak germicidal power. For reference, the total power application is shown, which is in this case approximately 3.7 times the germicidal power. If the previous example is continued for the 30,000 m^3/day (8 mgd) plant requiring 18 KW peak germicidal power, the equipment purchase cost would be $240,000. O & M costs are estimated at $30,000/year, while total yearly costs (again, excluding support equipment), is estimated at $50,000/year. These cost curves were then used to develop unitary U.V. cost estimates based on flow.

The costs presented on Figure 10 are based on the assumptions of an influent fecal coliform density of 10^5 and a desired effluent fecal coliform of <200 MPN/100 ml. The costs associated with

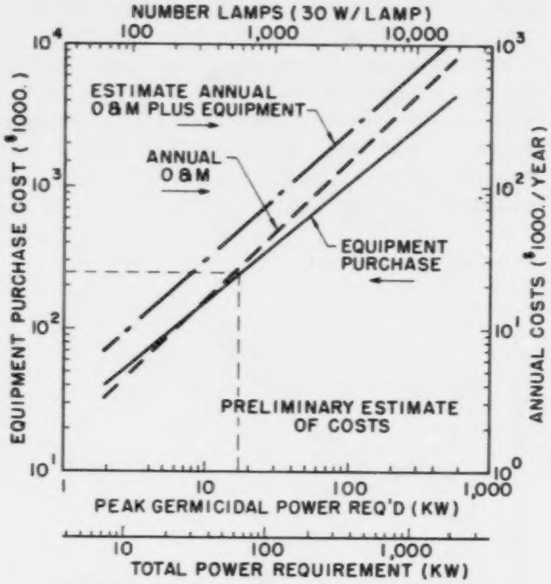

FIGURE 9. TOTAL POWER REQUIREMENT (KW)

DESIGN AVG. (mgd)	PEAK GERMICIDAL (KW)	O & M (¢/1000 gal.)	TOTAL (¢/1000 gal.)
1	1.0	0.6	1.4
5	5.2	0.6	1.2
10	10.0	0.5	0.9
40	43.0	0.5	0.9
100	100.0	0.5	0.9

FIGURE 10. PRELIMINARY ESTIMATE OF COSTS

the 19,000 m^3/day (5 mgd) are estimated at 1.2¢/1,000 gal. The cost for plants greater than 38,000 m^3/day (10 mgd) are estimated at 0.9¢/1,000 gal. (excluding amortization of costs associated with support structures). As discussed earlier with the projected design nomograph, the design, under similar water quality conditions, is sensitive to expected influent coliform densities. It follows that the costs associated with the process must also be sensitive to these conditions. The factor applied to costs for each log increment is estimated at this time to be 3.5.

It must be stressed at this point that the costs discussed above are preliminary and exclude certain capital cost items. It is anticipated, however, that the unitary U.V. disinfection costs will fall in the range of 1 to 4¢/1,000 gal. (under the operating conditions described above). These are very cost-effective when related to similar cost estimates for chlorination/dechlorination (4-8¢/1,000 gal.), and ozonation (10-15¢/1,000 gal.). Several factors will impact U.V. disinfection efficiency, and in turn the costs associated with the procedure. These include influent densities, water quality (studies at Northwest Bergen have dealt with a consistently good quality effluent), and photoreactivation, which is discussed below.

Photoreactivation

The lethal and damaging effects of ultraviolet light on bacterial cells are the result of absorption of ultraviolet energy by the cell's genetic material, deoxyribonucleic acid, or DNA. The damage incurred by this absorption is the dimerization of thymine, one of the basic components of DNA. This dimerization can be reversed by subsequent exposure of the irradiated cell to a variety of energy wavelengths in the visible light range of approximately 310 to 500 nm. This phenomenon is called photoreactivation. Research has shown that substantial increases in cell viability do occur by this mechanism in laboratory controlled experiements with pure cultures.

Studies to determine the significance of photoreactivation in recovery of U.V. irradiated coliforms at Northwest Bergen have been conducted throughout the program. Preliminary testing during April and May evaluated recovery as a function of time, and also investigated recovery in-situ by sampling in the chamber itself (see Figure 2).

During the June thru August detailed experimental program (which has been the subject of this paper), static light/dark bottle tests were utilized to evaluate photoreactivation. Aliquots of 100 of the 119 U.V. irradiated samples drawn were collected in borosilicate bottles and placed in a water bath held at the sampling temperature. The sample was then exposed to natural daylight conditions for a period of one hour (which had been determined as the approximate time period over which maximum repair would have been accomplished). After the hour exposure, both total and fecal coliform densities were determined. Approximately 25% of the samples analyzed by this procedure were also held for one hour in dark bottles as controls. There was no measurable difference beteen the coliform densities in the dark bottles after one hour, and the densities observed immediately after irradiation.

Figure 11 presents the results of the photoreactivation analysis as a correlation of log reduction and log dosage. Regression lines were computed by the least squares procedure and are shown on the Figure. Regression lines computed for samples analyzed immedately upon irradiation are superimposed upon the photoreactivation results. The obvious shift is due to the repair of injured cells.

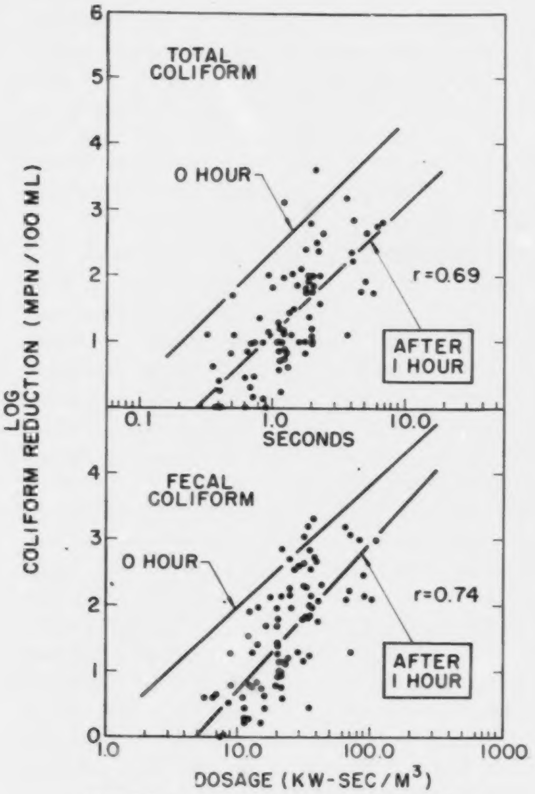

FIGURE 11. REDUCTION AT ONE HOUR

Similarly, the regression lines computed for log effluent coliform versus log dosage at 0 hour (immediately after irradiation) and 1 hour (exposure to visible light) suitably demonstrate the impact of the photoreactivation phenomenon as shown on Figure 12. Repair accounts for fairly consistent one log increase in effluent total coliform densities. The increase in fecal coliform levels varied from approximately a one log increase at the lower dosage level to 0.6 log increase at the higher levels.

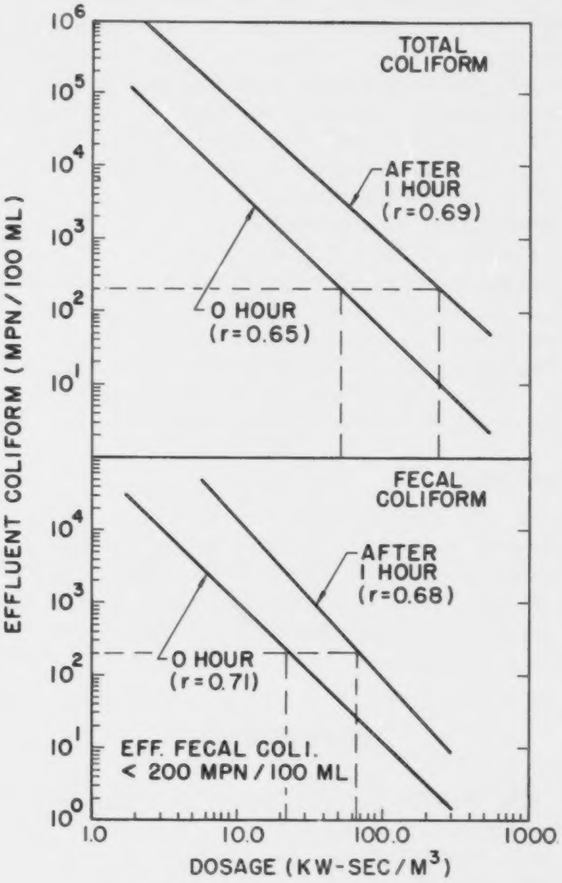

FIGURE 12. PHOTOREACTIVATION

SUMMARY

The results of the U.V. disinfection demonstration project to date have suggested the following preliminary findings:

The thin film, gravity flow disinfection unit has been found to provide effective treatment with low maintenance over a four to five month period. It is flexible in its operation and mechanically simple. The wiper mechanism has had approximately 3,000 hours continuous operation with no apparent degradation in cleaning efficiency. The lamps have typically operated an average of 2,000 hours.

At fecal coliform influent densities of 10^5 MPN/100 ml, a dosage requirement of 35 KW-sec/m^3 is suggested to meet effluent criteria of <200 MPN/100 ml. This, of course, relates to the water quality conditions at the Northwest Bergen County plant. Costs, without consideration of support structures and ancillary equipment, are estimated to be in the order of 0.9 to 1.4¢/1,000 gal. Both cost and disinfection unit sizing are sensitive to the influent density levels by factors between 3 and 3.5 per log increment in coliform density.

Photoreactivation has been evaluated, and significant increases (0.6 to 1.0 log increments in fecal coliform levels) have been noted after exposure of irradiated samples to visible light. The implication of repair (or aftergrowth noted in other disinfection procedures) may be significant when related to unit sizing and costs.

AUTHORS

Mr. O. Karl Scheible is a Senior Engineer at Hydroscience; Ms. Gloria Binkowski was a Microbiologist with Hydroscience at the time of this study and is now affiliated with Rutgers University; Mr. Thomas Mulligan is Technical Director at Hydroscience.

DISCUSSION

MR. WHITE: Did you run a dye study curve on your contact in the unit?

MR. SCHEIBLE: No, we did not.

MR. WHITE: Are you going to?

MR. SCHEIBLE: It is not in the program. I find some difficulty there in trying to get any accurate measures even on a dye study in the order of fractional seconds. That scares me.

MR. WHITE: Don't you think it is better than nothing?

MR. SCHEIBLE: Probably. We did some velocity profiles across the units both horizontally and vertically to see if there was any dramatic changes in the velocity on the effluent side of the unit, and we did not see any. No evidence of real short-circuiting through the unit.

MR. WHITE: In other words, you are assuming there is no short-circuiting so you simply do a V over Q to arrive at your seconds. Is that right?

MR. SCHEIBLE: That is correct. It is a void volume over the flow.

MR. WHITE: Very unscientific.

MR. DeSTEFANO, *Riddick and Associates:* I noticed that you had to chlorinate. I visited the plant, and I noticed that the injection point was probably about ten feet beyond the unit. Now, did you check to make sure there was no back diffusion of chlorine? Did you take chlorine residuals in your samples?

MR. SCHEIBLE: Yes, we did.

MR. DeSTEFANO: There was no chlorine residual?

MR. SCHEIBLE: No. That was moved, incidentally, down to 30 feet just for safety purposes.

MR. DeSTEFANO: Also, this idea of calculating dosage is very convenient but you seem not to be taking into account UV absorbance, and it does not matter how much power you are applying to the liquid if the liquid is going to be absorbing some of the power. All you are concerned about is how much power is hitting the cell, not the liquid, because you are not disinfecting the liquid. You want to kill the cell.

MR. SCHEIBLE: I agree. I do not know if I can call it a drawback to this study, but we have dealt with a very consistent quality of water, and due to that we have not been able to really correlate anything. We have tried multiple regressions bringing in the idea of water quality but due to the consistency of the water over a three month period we are not really able to correlate UV absorbance.

MR. DeSTEFANO: A comment about the quality of the water. The effluent quality was very good, but there were also some very large floc particles when I saw it. I do not think the plant was down. I think that is a normal condition, very light though. You probably do not have a high suspended solids, but in terms of volume you do, and I am not sure that when that thing goes zipping through your unit at 2.9 seconds that a little bug inside one of those floc particles is going to get hit.

MR. SCHEIBLE: We did have a condition of floating material, large clumps. This could be brought on by two operating conditions. During a portion of this study at night we dropped the flow back down and dropped the lamp power down just to conserve. If you recall the slide showing the installation of that unit, there is effectively a dead space in the influent channel to the time when it hits the UV unit. The UV unit sits about six feet up. If you drop the flow, there is material that tends to settle in that chamber. If you come in in the morning and crank up the flow, it would scour the bottom and push the particles through, and we could only exercise the caution that sampling would not be performed during that flush period.

MR. DeSTEFANO: When I saw the plant it was about two in the afternoon, and someone did take a sample as we were there, and you could actually see the floc going through the unit. They are not really floating. They seem to have about the same density as water in that they stay somewhat submerged on the bottom, not on the top, and you could see it just ripping through.

MR. SCHEIBLE: One other point I was just going to make is that plant is nitrified. Thus, they do have problems at times with floating sludge in their secondary clarifiers. They also pull off a lot of algae from the secondary clarifier.

MR. DeSTEFANO: One last question. Did you correlate UV light intensity with voltage?

MR. SCHEIBLE: It is linear.

MR. DeSTEFANO: But you cannot go all the way down. I understand that once you get down about 40% the lamps will go out.

MR. SCHEIBLE: The lamps start shutting off on you at about 40 or 45% of full power.

MR. VENOSA: Don't forget that some floc will go through a chlorine contactor too.

16.

ROUNDTABLE DISCUSSION OF ULTRAVIOLET LIGHT DISINFECTION

Participants in Ultraviolet Roundtable

Albert D. Venosa, Moderator
U. S. EPA, MERL-Cincinnati

1. Harold Wolf
   Texas A&M University
2. Kent Aldrich
   University of North Carolina at Chapel Hill
3. M. Elliott
   University of North Carolina at Chapel Hill
4. O. Karl Scheible
   Hydroscience Associates, Inc.

MR. GOLDRING, ***Orion Research Incorporated:*** I have what are in essence, I think, one question and a comment on Mr. Aldrich's work. His results, I believe, are much easier to understand if one assumes that he is killing all of the micoorganisms in 99% of the solution and 1% of the solution never sees the UV at all. As he mentioned he had difficulties with the bypassing in his equipment, so essentially he is measuring the bypass rather than any effect of UV, I believe.

The other thing is that in the actinometry he did not mention the transmittance of the solution that he used for his actinometry. If it is low, that is, if he has a high absorption coefficient, he is measuring the output of the lamps and not the exposure of the solution. In other words, to measure the exposure of the solution and compare the microbiological data he should be working at an absorption coefficient that is similar to that of the untreated waste water. In that case he would at least be seeing the way the UV is absorbed by the solution when its absorption coefficient is approximately the same as it is when he has microorganisms present.

MR. ALDRICH: That is correct, basically. What we started with was the high concentration actinometry and with the concentrations that high the solution absorbed the UV within a depth of only 3 to 4 mm. What we are interested in doing now is lowering the concentration of the actinometry solution, as you suggested, and we hope to be able to obtain a dosage which is even more meaningful.

DR. JOHNSON: I think you have to put this dosage business in context. The way that it has been classically done and that the standards are assigned today is so many microwatt seconds per square centimeter, and as we look at dosage, of course, the amount of UV added to the water can be measured just simply on the basis of inches of lamp and the way the manufacturer rates his thirty watt bulb, which is essentially what was done with the practical study in Northwest Bergen County. The next step is to measure the real output of the lamp with actinometry, and that is what you are measuring. You are not measuring the germicidal dose to any one microorganism, because he sees all of the UV absorbing fluid draining. He sees the wall over there that may be stainless steel; it may be aluminum that is reflecting the UV off the wall and back to him, and they are multiple lamps. These units are six and nine or so lamps apiece, not one lamp, so that you just cannot take the intensity or distance and measure the decrease in intensity with distance, because there are several lamps influencing one spot within the unit. The classical work that Hill did was with a two-lamp unit, and that is what the 16,000 microwatt seconds per square centimeter standard of UV disinfection in drinking water is based upon. So we are taking this a bit at a time. Our eventual goal, of course, is to measure how much UV energy comes to the microorganism, but we are a long way away from that as you point out.

DR. ROSEN: I have a question about the possibility of free radical formation and molecular rearrangements. After all, we are apparently absorbing UV and disrupting bonds, especially with high intensity UV. I wonder if anybody has any comment on this, or did anybody look at this, or what kind of theoretical probability or possibility is assigned to it?

MR. VENOSA: Bob Jolley tomorrow will be giving a presentation where he has evaluated the effect of chlorine, ozone and ultraviolet light on secondary and primary effluents. So he will answer your questions tomorrow.

DR. JOHNSON: The energy is fairly low, 254 nm, not a lot of energy. It is not down to the 180 nm region where you get significant bond disruption. It would take a fairly weak bond to come apart at 254 nanometers.

DR. ROSEN: Yes, but you know you have a spectrum. The 254 nm line is the major line.

DR. JOHNSON: Well, the mercury lamp puts out about 90% of its energy or more at the 254 nm region. There is some energy at 186, and then there are a few lines up in the visible region, but they are very small. These lamps are tuned to the mercury line at 254 nm.

MR. DeSTEFANO, *Riddick and Associates:* I would just like to explain my dual nature here. I am representing a consulting engineering firm that is also, in turn, representing the Village of Suffern in New York, which has now a discharge permit for class A waters. They do not want any chlorine residual. We looked at various alternatives and in the most recent facility plan we are recommending a UV system. But the problem we are facing, as I see it as a consulting engineer (I am not myself an engineer; I am an undergraduate student at Princeton helping out for the summer) is when the engineer goes out and talks to the manufacturers, who seem to be the only ones up until now who had information, he gets very conflicting ideas, very conflicting dose recommendations, flow recommendations, and geometry recommendations. We talked about it. We got a little bit into it, thick film, thin film, batch, flow through, and I am really at a loss on how to handle all this. I know there are a lot of manufacturers here, and I won't be afraid to say it but . . . each one has a bad word for the other, and everyone thinks that they have the best system, and I was wondering if someone up there could give us a hint on what sort of things you want to look for when you are designing the plant. Do you just want to look at performance, or do you want to have a margin of safety, or do you want to just look at your counts, or do you want to look at the dose? I will leave it at that, and you can play with it from there.

DR. JOHNSON: Well, I am not sure what your question is, but . . .

MR. DeSTEFANO: I will clarify it. Could you just generally give something that an engineer could look at and say "These are my design parameters. This is my effluent quality. I want to have a certain type of ultraviolet disinfection system with a certain amount of wattage or a certain amount of dose or whatever method you want to use to measure the disinfecting power of the ultraviolet taking into account geometry", because obviously there is some difference in proprietary units available.

DR. JOHNSON: Well, current design is focused upon dosage, that is, microwatt seconds per square centimeter. It is conventionally measured in the literature, or as we have talked about today in terms of watts per cubic meter, which is essentially the same sort of thing just on a different basis in terms of power output from the lamp in germicidal power. In our studies we found that there was way too much lack of concern for flow characteristics. Most of the UV has been applied to drinking water. Drinking water does not usually have 10^6 microorganisms in it, hopefully. So they are not looking for 10^4 kinds of orders of kill like they are in wastewaters, and of course if you have a 1% short circuit in your contactor then you are never going to get much passed the 10^2 kill. Thus retention time is very important, and the flow characteristics of your contactor are very important. There has not really been enough considered in wastewater design in disinfection units.

MR. DeSTEFANO: What about the issue of ultraviolet absorption of the effluent?

DR. JOHNSON: That is, of course, the other side of the coin. What is the tradeoff between thickness? One person says you need a quarter of an inch and the next person says you need three inches. Basically what you are looking for is UV energy to the microorganism. Now, there is a lot of theorizing you can do, and I think that is where we are at this point. It is obvious that UV transmission at 254 nanometers is important, and if you have a high quality of waste which does not have much absorbance as they seem to in Northwest Bergen County, then I think they are kind of foolish to use a quarter of an inch of lamp depth, because they are throwing a lot of energy away by reflections off of the quartz tubes next to each other. Just heating the unit is where most of their power is going to go. They are doing a rather inefficient job when they work with a quarter of an inch film thickness, and an absorbance of .17, but it is theory you know. We do not really have a design parameter as yet to do a good job, but that is obviously very important. Harold said ammonia is a good parameter, but of course when you look at sewage treatment, microorganisms first go after carbon and then go after nitrogen, and so naturally the amount of organics is important in terms of the 254 nanometer absorption, and that follows that ammonia might be a good way of correlating on a control basis, but a much more direct basis rather than measuring TOC or ammonia or COD. It is just a measure of 254 nm absorbance

MR. DeSTEFANO: But what about a plant that is not running very well. I know the Northwest Bergen County plant is running under capacity, and it is producing a very good effluent, but what about just standard secondary effluent or bad secondary effluent such as the other plant in Bergen County, the Bergen County Sewer Authority plant, which, as I understand it, is rated at 40 mgd and is running at 70 mgd, something like that. I think that is where this unit was initially installed and they were not getting disinfection at all because it was essentially a primary effluent. What I want to emphasize or try to get from you is what kind of effluent characteristics do you need to make a valid application of ultraviolet light, and what kind of effluent characteristics coupled with what kind of geometry? Obviously, I know you do not want to recommend any particular manufacturer or any particular geometry, but outside of just ultraviolet transmission there has to be some other things that you can look at to try to verify that you are getting the energy to the organism.

DR. JOHNSON: We had a fairly low dosage at the beginning of our curve, 10,000 microwatt seconds or less which is generally rather small, and we were putting it through wastewater which had suspended solids in the moderately good range. It was not the best. This is a contact stabilization plant that runs pretty much up to capacity, and we were doing a good job with one inch depth of water through a conventional kind of unit. I guess what I am trying to convey is my skepticism that UV would work to begin at all on wastewater, and surprised that is has worked as well as it has.

MR. DeSTEFANO: What about something that you might not be able to measure except by ultraviolet-absorbance some sort of dissolved iron. I know in our village of Suffern there are a couple of industries that might be giving us something funny. Avon is one of them, and who knows what they are dumping into the line that might just absorb incredibly at 254 nm but the effluent characteristics by standard methods by turbidity would be looking good.

DR. JOHNSON: Yes, you cannot use the TOC, COD, suspended solids kind of measurements. You have to measure the 254 nm absorbance because iron is one of the best UV absorbers, and iron is not measured by any of the standard procedures. So if you have some iron coming down from an industrial waste you could completely destroy UV disinfection.

MR. VENOSA: Are you ready to hang your hat on the statement that COD and TOC do not interfere with UV?

DR. JOHNSON: I did not say that. I said you cannot just use those alone. You have to use the parameter that is important interacting with UV. Why measure TOC when you can correctly measure 254 nm absorbance. It just does not make any sense.

MR. ELLNER: To change the subject a little bit, there are two points I would like to make. One is, I do not want anybody here to get the impression that photoreactivation is a phenomenon that is limited to ultraviolet disinfection of organisms, and I think the point should be made that this repair has been studied and I think should be studied, and has been demonstrated with more conventional types of disinfection as well, and you can apparently establish increase of organisms through photoreactiviation.

MR. SCHEIBLE: I just want to qualify your statement. Photoreactivation is unique to UV. It is due to the presence of photoreactivating enzymes. I agree with you that other disinfection procedures should be looked at for "aftergrowth" or any other mechanism that increases the effluent quality.

MR. ELLNER: While we are on that subject too, I think some review should be made of the actual practicality of the conditions of photoreactivation and actual practice of a discharge into a receiving stream as clear as the Hudson River and things of that nature which offer tremendous amounts of protection from visible light available.

MR. SCHEIBLE: I will take issue with that too. If you look at a number of the streams that the UV or any disinfected effluent discharges into, those are small and shallow, and they are normally of high quality.

MR. ELLNER: The point I was trying to make is that it has to be related to the practical aspect. This concept of dosage probably highlights the major problem with UV from both the theoretical and practical standpoint. The researchers have no sure fire way of coming up with a dosage to quantify their results, and the engineering and specifying people have no way of describing a UV unit to meet a given design parameter. Now, there have been attempts to come close to that, and I would like to offer this because researchers are starting to report it, instead of something that is questionable in the measurement. There is no conventional UV dosage determination, at least you are not going to get a consensus on people in the field. There has been an attempt though to relate results to how many gallons per minute per unit length of ultraviolet source, and how many seconds retention time, defining, of course, the output of the UV source, because that is something that can be duplicated by the next researcher. I propose that as some thought that certain people have started to specify ultraviolet on that basis. I am going to add just one more thing. I do not want to monopolize this, but I represent an ultraviolet firm, and I make the statement now that we do not utilize this conventional estimate measurement of ultraviolet dosage technique. We have developed proprietary dye tests for determining ultraviolet, but we also have used some follow up approaches which I think we can reveal here, and I think should be under consideration, and that is a very simple scientific technique known as the bioassay, and essentially what is done is that you develop pure strains of organisms, plot the log reduction in relationship to the UV dosage which can be determined on a static basis, and then introduce your organism to the test unit, measure the log reduction coming out on the other end, and have a good cross check as to what the dosage delivered was.

MR. WHITE: I have a couple of quick questions I want to sneak in here. Number one is: you speak of slime on the lamp, and that is what I heard you say, Dr. Wolf and Mr. Scheible, and this is where the UV intensity is highest, right on the lamp. What is the nature of the slime? Is this bacteria? If it is, why isn't it killing the slime? That is question number one.

MR. SCHEIBLE: I did not speak of slime on the quartz tubes themselves.

MR. WHITE: You talked about the wipers. What is this stuff that you have to wipe off? Is it a zoogleal slime, or what is it? Does it have organisms in it?

MR. SCHEIBLE: There was an occasion very early in the study when we were still in the procedure of equipment shakedown and getting everything working properly, and it happened that the wipers were turned off for a period of days so we had no mechanical wiping system in operation, and what seemed to develop on it was a thin, white film coating, that essentially became baked on.

MR. WHITE: Just like a calcium scale deposit?

MR. SCHEIBLE: I do not know. It is baked on simply from the heat of the lamp. When we pulled the unit up out of the water to take a look at this, we experimented with several cleaning solutions and came on one and just sprayed it, turned the wiper on, the wiper scrubbed it off, put it back in the water. I think I stated at the end that the wiper mechanism itself operated over a period of 3,000 continuous hours, and we have not to date seen any deterioration in the intensity off the quartz tubes.

MR. WHITE: Okay. I was just curious whether or not it was some organism. The second question is: do you generate any ozone in the operation of UV systems?

MR. SCHEIBLE: Yes, it does in fact. As the study continues on into the winter, we may have to blow warm air between the lamps and the quartz sleeves to maintain the lamps at optimum temperature. We can then take that exit air, which becomes enriched to a degree in ozone, make use of it, and inject it into the front end of the system, and get whatever synergistic effects may occur. We are not sure what will occur.

DR. JOHNSON: We were interested in the ozone question, and thought that if there is oxygen in the water, why don't you get ozone in the water. The trouble is the water is very good at quenching the UV. The wavelength that produces ozone is 186 nm. Even if you can get 186 nm through your solution, which is doubtful in wastewater, the oxygen that is dissolved is not capable of producing much ozone. Most of the ozone comes from the air space that is around the lamps. These lamps have to be cooled. The operate very much at an optimum temperature. At slightly below or slightly above that temperature their efficiency falls off, so that it may not always produce the optimum 40 watt germicidal output. And what it puts out in the 186 nm region is mainly producing ozone in the air space.

MR. WHITE: An efficiency of 200 fecal coliforms per 100 ml, which you spoke about as the goal, is not considered disinfection in California. It is a long story, but I just wanted to mention that. The other thing is that some of the people I talked to in France, who have been looking into this, suspect that UV will cause mutagenesis in surviving viruses.

MISS ELLIOTT: I have found in my reading that there are mutagenic effects of UV light, and as far as the photoreactive ability of these organisms, it varies among the organisms of course, but mutagenesis has been observed.

MR. HEINSOHN: I was an old Princeton hat. Let me answer this young Princeton hat back here. First his Suffern plant is about fifteen minutes from where I live, and that plant is an operational disaster. The first thing you have to do over there is re-engineer that plant, because what comes out of it is just about what goes into it. I was called over to have a look at this place. I could show you some others in the country too where the lawns look like putting greens and a few other things, but what comes out of the plant is terrible. I just want to answer him this way, and say that from a manufacturing position of UV equipment for wastewater, we do have production systems on the line. Let me tell you all that the target in our business is to reduce any coliform count down to a count of less than 3. That is the target, not 200 per 100 ml but less than 3. Some states have that spec right now.

MR. DeSTEFANO: Can I just respond to save the face of Suffern? Admittedly the plant is now not operating at optimum. It was a WPA plant. It is old, and it is running at, I think, twice it design capacity. There is also a problem with inflow. There are a lot of other problems, but that is why we wrote a facility plan. The plan will be expanded, and we will be working with what we assume will be a good secondary effluent, and there is going to be some tertiary treatment, not filtering, but tertiary treatment for nitrate removal. I would just like to respond to my fellow Princetonian and maybe I will get some of these UV manufacturers again. The reason we got involved in this is the project engineer told me to get on the phone and start calling people, and it is a very strange way to try to get involved in a subject you do not know much about. I called Mr. Heinsohn. We got his name from some people up at New York EPA, and we talked about the plant over the phone. He took it upon himself to visit the plant and then make his design recommendations. Really at that point we were just feeling out manufacturers there. We are a bit further now, and the facility plan is written, but we are still not at a point where we have written a spec or anything. The design still is in the future, and that is why I am here to find out what sort of design considerations we have to make before we go into the construction phase.

DR. GREENBERG, *Department of Nuclear Engineering at The University of Cincinnati:* I would like to offer a suggestion to those who are working with these photo-chemical reactors. We in chemical engineering characterize chemical reactors by residence distribution studies, and I am rather suprised that I have not heard or seen any reports of such studies among the presenters today. I suggest that that would be an easy way to characterize the circulation and distribution around the photochemical lamps. I would also like to comment on some parallel work that I have been involved in using coherent light for disinfection of various microorganisms. In scanning from the near UV, 265 nm being the peak absorption for DNA-RNA, individual species of organisms absorb in a very characteristic pattern. At least this is our preliminary analysis, and this extends, I guess, through the visible and into the near IR. You can promote growth by irradiating at different wavelengths. You can inhibit growth at the same wavelength by looking at fingerprints of different organisms and overlaying these fingerprints, and irradiating at a selected wavelength, using again coherent light or laser light, very selectlively. We have had some success in doing this.

I would also like to call attention to some work that was done by a colleague at the Applied Physics Laboratory, Johns Hopkins in Maryland about two years ago. Dr. John Parker irradiated a sample of seawater or wastewater in a simple batch experiment with infrared light from a CO_2 laser, and discovered that by pressuring the sample with oxygen he was able to generate the oxygen singlet, which has the same germicidal activity as ozone. He was very effectively able to reduce the microorganism count. I suggest that this is something that perhaps ought to be considered.

DR. JOHNSON: Is this in water?

DR. GREENBERG: In water, yes. Also the penetration as I understand it at these high frequencies, that is, in the near UV, is on the order of a few mm, at most, and I am surprised that we are talking in terms of inches. I think the absorption is greatest . . .

DR. JOHNSON: We measure directly the UV absorbance at 254 nm, the wavelength of interest, and the absorbance values come out to be relatively small. In operating an actual plant, your values were up to three-tenths of an absorbance unit. We did not find anything like one absorbance unit, which would be 10% of the material going through a one centimeter thickness. These wavelengths are not all that far down in the ultraviolet.

DR. GREENBERG: Are you suggesting then that there is very little absorbance as a function of distance out to several inches?

DR. JOHNSON: Well, we measured. I am not suggesting anything. I am telling you what the measurements are. The measurement said in an actual wastewate that the absorbance was less than one absorbance unit, and if you look at Beer's Law relationships, that tells you that 10% of the material got through one centimeter of fluid, 90% got absorbed, and the absorbances were well below one. Most of them were around 0.17. That is in one cm of the fluid; if you convert that, say 80% goes through.

DR. GREENBERG: 80% goes through, and this is wastewater?

DR. JOHNSON: In wastewater.

DR. GREENBERG: Is this information available? Is it published?

DR. JOHNSON: There is published data. It goes back to some works that were done by General Electric. Lukeisch did work back in the fifties or earlier than that. He looked at UV absorbance of wastewater, drinking water, and a number of different types of waters in the country. This was typical.

DR. GREENBERG: But as one goes down in frequency towards the IR, the absorbance, the penetration let's say is much much greater.

DR. JOHNSON: Yes, if you go down to 200 now everything absorbs.

DR. GREENBERG: But in the near IR you can penetrate through several feet of dirty water which is what Parker did in his study.

DR. JOHNSON: Let me just reinforce something that was said by the last commentor. There has been far too little concern about the detention time. George showed some data on detention time, and talked about a t_i, the initial breakthrough of material in a dye study. Even the better of the two units that we looked at where the mean detention time was not significantly different from the volume divided by the flow rate, the t_i value was something like six seconds when the mean detention time was twenty seconds. Now, that is at a level that you can read off this curve of ours which is maybe at 1%. If you are thinking about wanting to get four or five log reduction, then you ought to call t_i where you get the first 0.01% or four logs, and that would be a rather short contact time even for the better of the two units. So there is far too much concern about dosage, microwatt seconds per square centimeter in ultraviolet disinfection of wastewater today. There has not been enough concern about detention time, the main point of our work.

MR. WOOD: On that subject, the detention time can be two ways. It can be long or it can be short. We are paying attention to the short retention times, because that marries to a piece of equipment that practically fits a sewer plant operation. You do not want a monstrous piece of equipment that takes two acres of land to put it on. What you end up with if you are talking about long retention time, is hydraulic loss problems. You get into flow conditions. You get into a wide gradient of exposure as you go away from the source. These are some of the things that are of concern. We promote thin film, short retention time. So we are paying attention to retention time on the short side.

One other thing I would like to clear up. It was brought up here that this particular unit at Northwest Bergen Sewer Authority was installed at Bergen County. That is an error. The unit that was installed in Bergen County was the first prototype unit that we built. It had around a 6 mgd flow capacity. We did get performance on that unit at a flow condition where the plant was twice capacity, and effluent quality did not meet standards, had suspended solids up in the two hundred range, color and turbidity up in the fifty to hundred range. We met the 200 per 100 ml standard in that installation at a flow rate of about 5 mgd under those conditions. At that point, we made the decision that we just did not want to have that particular system associated with that type of plant. At the time we were conducting talks with EPA, they felt that if ultraviolet was going to be a successful disinfection alternative you did not really want to evaluate it at a very poorly run plant. It would mean that you could run your plant anyway you want, just place a UV unit at the end of it, and it overcomes anything you might do in the operation of your plant. That was our reason for going to Northwest Bergen where we had a more model plant, and we felt it gave UV a more representative test also. I would subscribe to the fact that the plant should be run well and if it is run well, ultraviolet is a tremendous disinfection means.

The other point I would like to make is that I think we have the tendency in statistical analysis to look at log reductions and maybe not relate them to actual conditions, and I would just like to point out a layman's approach to the testing we are doing at Northwest Bergen. We have had ultraviolet disinfection system in there that has been running around four months, full plant flow, pretty much somewhere between 80 and 100% power. In four months of day-to-day testing, twenty-four hour operation, we have never on a single test reached a 100 per 100 ml count. Now your curves say that that unit cannot do it. I want to point out that the operator's standard is 200 per 100 ml. That is the performance standard he has to meet, and if the unit does it, in my opinion that unit performs. Statistical analysis be damned!

DR. SCHWARTZ: I have some questions. First of all, UV is known to induce moleclar rearrangements in DNA. Ultraviolet is known to induce transformation, mutagenesis, and carcinogenesis when organisms are irradiated, and molecular rearrangements do occur in the DNA, not directly due to the ultraviolet but due to the repair processes within the cells and viruses. The thing that I want to bring up that is an important point is not the photoreactivation, which is a rather benign sort of repair, but the dark repairs which are error prone. Has anyone who has run these UV disinfection facilities done any transformation studies of the viruses that they inactivated?

MISS ELLIOTT: As I mentioned earlier, we were working with just total coliform evaluations, so I have not.

DR. SCHWARTZ, *Deltech:* I have some questions. First of all, UV is known to induce molecular rearrangements in DNA. Ultraviolet is known to induce transformation, mutagenesis, and carcinogenesis when organisms are irradiated, and molecular rearrangements do occur in the DNA, not directly due to the ultraviolet but due to the repair processes within the cells and viruses. The thing that I want to bring up that is an important point is not the photoreactivation, which is a rather benign sort of repair, but the dark repairs which are error prone. Has anyone who has run these UV disinfection facilities done any transformation studies of the viruses that they inactivated?

DR. JOHNSON: I think I would like to comment about this "super oxygen". We went through a nascent oxygen hulabaloo with chlorine years ago. These compounds like singlet oxygen and super oxygen are very unstable. Their lifetimes are very short. One of the major problems with disinfection from a microbiological point of view is to get the disinfectant to the vital site within the microorganism. In fact, many disinfection processes, and ultraviolet not being one of which, are transport limited. The rates of the disinfection process is more a matter of getting the disinfectant to the organism than it is getting the molecular reaction to occur. Ultraviolet is not like that. It is definitely going right to the active site, which makes ultraviolet kind of unique among disinfectants. But the tremendous amount of energy required to get a singlet oxygen or super oxygen is one thing that mitigates against that approach, and also the other is getting that form into the microorganism.

DR. OLIVIERI, *Johns Hopkins Univ.:* I wonder if both of the gentlemen from NW Bergen County might comment on the frequency in which they met the 200 fecal coliform level, taking into account the factor of ten that they observed for photoreactivation, be it light or dark reaction. What percentage of the time did you meet that taking into account reactivation?

MR. WOOD: I can answer that. We ran our own photoreactivation studies. We disinfected the downstream chlorination tank with 5 ppm overnight dosage of that downstream tank. We then drained that tank down with the UV in full operation, full power, and ran until we got zero chlorination in the entire downstream contact tank. We had a time basis to go by. We then continued to run 100% power on the UV and then took ten minute up to seventy minute samples in the channel, downstream from the UV and zero chlorination. We found, I think, good correlation between what Dr. Johnson got and what Karl got. We got a one log increase basically, 1 to 1.2 logs. Again I have to relate this back to the real world. At the time we had 20 fecal coliforms/100ml coming out of the unit with roughly 10^5 coming in, and we had a 25 count seventy minutes downstream. Now that, too, is a one log increase, but it is not anywhere near 200.

DR. OLIVIERI: A factor of ten increase is going to bring you from 25 up to 200.

MR. WOOD: We had less than that. We went from 20 to 30.

DR. JOHNSON: Well, of course, we saw some dechlorination data yesterday that had two logs of regrowth. There was a question about whether it was really growing or whether it was just coming up from the sludge at the bottom of the tank, but UV is not unique I guess.

MR. SCHEIBLE: With the data that we are reporting I could not make the simple calculation of percent time that we met the effluent criteria of 200. I was more concerned about relating that unit . . .

DR. OLIVIERI: That is going to be the ultimate . . .

MR. SCHEIBLE: I agree, but most of our program to date has been to evaluate dosage levels relative to disinfection efficiency, and in my mind . . . correct me if I am wrong . . . when an engineer comes in and he wants to design a system, he is going to know what the relative characteristics of his influent are, and he can then relate a dosage to a log reduction scale.

He can get an idea of what the expected effluent will be. That has been the thrust of our work to date. Now, I can go through that calculation, but it would be a difficult one in that we varied the operating conditions of that unit considerably.

DR. OLIVIERI: Once you put in the lamps how much flexibility do you have to overcome a poor design?

MR. SCHEIBLE: Considerable flexibility. The unit itself has the capability of shutting off banks of lamps. Now in my mind the ultimate design of this would be simply that you rate the power input to the flow. So you vary your power with your variation in flow through the day. You can tone down during low flow conditions at night or whatever. It also has the ability to turn off banks of lamps so you can vary . . .

DR. OLIVIERI: Well, the situation that I am concerned about is if you have to add banks of lamps.

MR. SCHEIBLE: If you design your system properly that should not happen.

MR. SCHEIBLE: I do not know how else to answer your question. It all comes down basically to a correct design of the system, and you should be able to handle what your expected conditions are in the near future. That is all I can really say. And photoreactivation, to answer that question, from the data that we showed over a variety of dosage levels, we got a fairly reasonable correlation which showed an average of one log increase over effluent conditions.

DR. OLIVIERI: This is precisely what I want to get at. That is an increase, and you are showing me that you are reducing to say a fecal coliform level of one hundred. If that is going to go through a one log increase, I am going to have, coming out of the plant or at some point downstream, a fecal coliform level equivalent to a thousand.

MR. SCHEIBLE: That is correct.

DR. OLIVIERI: You have not met the requirement then. That is the point I am making.

MR. SCHEIBLE: Let me make two statements on that. First of all, the UV equipment can achieve a level which, if you even take into consideration photoreactivation, will achieve effluent criteria. I would love to throw it open to the floor here to anybody who has any comments, and I wanted to bring it up yesterday and Al did. What is the implication of this after-growth and what is the implications of photoreactivation?

DR. OLIVIERI: They are two different processes. They are two distinctly different things.

MR. VENOSA: It does not make any difference. There is still the same effect. If you look at Kent Aldrich's data, he showed that when he achieved three log reduction he got about a one log photoreactivation, but when he achieved a four log reduction he did not get any photoreactivation. So I think possibly the answer might be simply stricter coliform standards.

MR. DeSTEFANO: I think the answer might be that it is not when you are getting a high ultraviolet dosage like you were in that second unit that maybe the mechanism and the site of action is different, and that it is not the DNA dimeration because if you are getting photoreactivation obviously it is not a resurrection happening. The cell never died. It was possibly altered in some point.

So my concern, somewhat similar to the gentleman from MIT, is that (and this is sort of my other nature as a Princeton student interested in public health rather than an engineer because I am going to throw a pipe wrench in the works) it is possible, I think theoretically, that what you are doing with the ultraviolet, especially at low radiation similar to the dosage used by biologists to study mutagenesis is you are somehow just stopping the cell from its ability to reproduce. It can catch up with that in photoreactivation. So it starts to reproduce again in your lactose. Also I know that (I have seen some references in the literature) one of the first uses of UV was as a mutagen, and it changed resistance of a bacterium to an antibiotic. I think it was streptomycin, and I think it was the same wavelength that was used. I just saw the reference to the article or the abstract. I have not read the whole article, so I am not going to hang my hat on that, but I would suspect (and I do not think any of the work done in Bergen County or the work done in North Carolina has taken this into account) that possibly at a certain dosage of UV, what you are merely doing or could merely be doing is inhibiting the cell's ability to ferment lactose. If that is done it is not going to show up on your test unless you are doing total counts too, because (I have worked with the MPN method) if it does not ferment lactose you are never going to see it.

MR. VENOSA: The cell's ability to ferment lactose is controlled genetically anyway.

MR. DeSTEFANO: But aren't you altering the genetic structure?

MR. VENOSA: Yes.

MR. DeSTEFANO: But could you also alter the genetic structure in a way that would inhibit its ability to ferment lactose?

MR. VENOSA: Yes you could, and you could also produce other types of mutations such as antibiotic resistance too.

MR. DeSTEFANO: I do not know if any of you are familiar with some other things that I have come across in my search of the literature of actions of UV light, one of them being its action on the membranes of the lysozomes and inducing lysis. It also induces lysis in another way, and I am not that versed in this, by releasing the coliphages in the *E. coli* and they all come out from wherever they are hiding and go with the cell. Again I just see those references. I think it was discussed in Strickberger's Genetics.

MR. VENOSA: I hardly doubt that that is going on in this system. Microbiology literature is replete with that type of information. It has been known for many years, but I do not think that is what is going on here.

MR. DeSTEFANO: How would you define its action?

MR. VENOSA: The mechanism of action for UV has been known for twenty-five years and it is the absorption by DNA and RNA. That is the primary effect. Now, certainly there are other secondary effects, but the primary effect is the absorption by the nucleic acid.

MR. DeSTEFANO: Now in doing my homework, just this section on UV says that you can get thymine dimeration, the DNA is altered, or you can get also I think uracil dimeration in the RNA. Now if the messenger RNA gets through and uses it as a decoder to produce the protein before the cell actually dies, you are getting a different code than the one that was originally in the cell.

MR. VENOSA: You are not going to get translation if the messenger RNA is blocked.

MR. DeSTEFANO: If you have only produced one dimer, why can't it go through. What is stopping it?

MR. VENOSA: By the very fact that the dimer forms it blocks further replication. You cannot get it. It is genetically impossible.

MR. DeSTEFANO That is not what Dyson implied. I am sure you know much more about it than I do.

MR. VENOSA: I think we are getting beyond the subject of this conference.

Why don't we call for one or more questions from the audience.

DR. ROSEN: I would like to ask a question of Mr. Scheible. I am sorry, it is practical.

You indicated that based on this particular design in this wastewater that the cost of a system for one additional log increase reduction would be on the order of a factor of four from the number that you calculate. Is that correct? Total cost?

MR. SCHEIBLE: Three and a half. I would also state that these are preliminary cost estimates.

DR. ROSEN: Okay. This gets back to some of these questions about where you sample, and whether you are looking at a sample after photoreactiviation or not in terms of what you count and what you do not count. If you are talking about a difference between a 200 fecal coliform, assuming you are starting at the same level, and a 2.2 total coliform in California, you are talking about a factor sixteen in the cost or 9, 10 or 12, somewhere in that range, based on these preliminary estimates, and I think that is significant and has to be considered in the whole evaluation here.

DR. JOHNSON: I might say that in my judgement with that particular wastewater this unit is killing the gnats with a sledge hammer. They are putting in a very large dosage of UV into the sewage of the type that was being treated in Northwest Bergen County. In other words, I think we are at the stage in ultraviolet disinfection we were with ozone five years ago. UV is just getting started, and I think that ultraviolet disinfection can do a much better job as we learn more about the process. It is going to get better, and I think we are going to have trouble if we continue to talk about ultraviolet disinfection in terms of suspended solids and conventional kinds of wastewater measurement parameters. It does not make any difference whether your suspended solids are 5 or 500. What is important is how much UV transmission you have.

DR. HILL, *Louisiana Tech. Univ*: A couple of the speakers have mentioned that the presence of iron inhibits UV disinfection. I am wondering if someone can put a number on this. I am thinking of a relatively small community that uses ground water for drinking water with just chlorination, and it comes out of the ground close to the public health service maximum of 0.3 mg/l, and there is some contribution in the distribution system from corrosion and iron bacteria in some places.

MR. ELLNER: Just as Dr. Johnson mentions, even with iron you cannot take a number and say 0.3 or 3 ppm of iron is going to inhibit UV performance. You have to relate it to UV transmission. From a practical standpoint and not taking up everybody's time. I will give you a copy of a study done here in Cincinnati at the Taft Sanitary Engineering Center examining the effects of iron, color, turbidity, but relating it to UV transmission. In their studies for your specific question, three parts per million of iron did not significantly affect the performance of a specific UV design parameter. One other point I would like to leave you with. When you review this discussion, and you take figures, they are not absolutes. For example, Mr. Scheible's economic figures, his cost figures, relate to a particular configuration that he studied. Now it is possible that there might be certain UV approaches that can cost three times as much, and there might be other UV approaches that can cost one-third. The suggestion I make is: keep an open mind and realize what we are discussing here is the broad brush stroke. Don't either dismiss or accept something based upon specific statements that are made today. I think we are creating the background for the picture, but a lot of things still have to be filled in.

DR. ROSEN: Getting back to using this UV absorbance as a measure of design or acceptability or efficiency of use of UV in disinfection, we are still talking about things like color and turbidity and things that we see in visible light. There must be a lot of UV studies done on wastewater historically in terms of trying to identify compounds, etc. My question is: Does anybody know if we do have some sort of range that we might expect based on lots of different kinds of effluents to try to get a feeling of where this fits? The things that have all been studied so far are pretty high quality relative to nitrification and some of the other things.

MR. REYNOLDS: *Aquafine, Inc.:* Most of the work done, and you can find this in Photochemistry and Spectroscopy by Simmons, shows general wastewater at 254 nm to have an absorption coefficient between 0.13 and 0.2. Typically that is going to take you down to about 60% transmission in a one centimeter cell. Now, there is also some recent data in Photochemistry and Photobiology where they are doing specific work on absorbance with anilins and other compounds which do have a tremendous absorptivity of UV at 254 nm. To elaborate on the iron, if you go to 0.3 mg/l iron, you are going to find tremendous fouling in the UV system very quickly because of the thermal differential on the quartz in the wastewater. Typically it tends to play down on the quartz jackets, thereby having to have some type of either mechanical or chemical cleaning in the system.

MR. WARRINER: *CH_2M Hill:* This may not be a response to Harvey Rosen's comment. I ran across proprietary equipment a couple of years ago in Britain for continuous measurement of TOC, which involved an absorption cell at 254 nm, and I wonder if it is used at the other site from the other point of view if that is not a possible source of this kind of data, and I puzzled about that because I continued to hear suspended solids, other visible light parameters used. I guess it is a question for Dr. Wolf.

DR. WOLF: I am familiar with the fact that in German drinking water practice there is some application made to measuring UV absorbance on a continuous twenty-four hour basis. I am unaware of this type of application in wastewater. Certainly from what I have found myself this would be a terribly fruitful path to follow with respect to getting some kind of handle on the quality of effluent with respect to organic properties. I really think so.

SECTION 5. OZONE

17.

COMPARISON OF MPN AND MF TECHNIQUES OF ENUMERATING COLIFORM BACTERIA IN OZONATED WASTEWATER EFFLUENT

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ABSTRACT

In 1976, an amendment to the Code of Federal Regulations (40 CFR Part 136) (12) was adopted establishing use of the Most Probable Number (MPN) Technique as the method of choice for enumerating fecal coliforms in chlorinated effluents in enforcement situations. From this it would follow that the MPN test would be the method of choice for all disinfected effluents regardless of the nature of the disinfectant. This study was conducted to determine if the more rapid Membrane Filter (MF) Technique could be used to enumerate coliform populations in ozonated effluent. A comparison was made between the two methods at different applied ozone dose levels. No significant difference between the two methods was found regardless of the magnitude of the applied ozone dose. Thus, either the MPN or the MF technique may be used to quantify coliform populations in ozonated secondary effluent. However, since the Code of Federal Regulations specifies the MPN as the method of choice in enforcement situations, compliance with the USEPA Environmental Monitoring and Support Laboratory's (EMSL) equivalency testing program would still be required to get approval to use another method such as the MF.

INTRODUCTION

Several researchers have reported that the membrane filter (MF) technique significantly underestimates the true population of coliforms in chlorinated water and wastewater samples when compared with the multiple tube fermentation (MPN) procedure (3,4,6,8,11). The reasons for the discrepancy between the two methods are unknown for certain, but one possibility is that the surviving organisms, which have already undergone one environmental stress (i.e. exposure to chlorine) are rapidly subjected to a severe temperature stress when transferred to the 44.5°C water bath. This, plus the fact that nutrients in the MF medium, which must reach the organisms on the membrane filter by diffusion, are presumably less readily available to the organisms to allow them to survive this stressed state (7). The MPN presumptive broth medium provides a more favorable environment for sublethally injured cells since it is non-selective and does not expose cells to the additional temperature stress. (5,9)

Ozone is rapidly gaining ground in the U.S. as a viable wastewater effluent disinfection process alternative to chlorination. Several ozone disinfection systems are on stream with many others under construction or in the design stages. Since the amendment to the Code of Federal Regulations (40 CFR, Part 136) (12) was adopted establishing use of the MPN procedure as the method of choice for enumerating fecal coliforms in chlorinated effluents in enforcement situations, it follows that the MPN test would be the method of choice for all disinfected effluents regardless of the nature of the disinfectant. Since the MF technique is rapid, simple, and more economical than the MPN method, it is desirable to use it rather than the MPN method if comparability can be demonstrated. To this end, this study was performed.

MATERIALS AND METHODS

Media

The MPN and the MF procedures were performed according to Standard Methods (10). In the MPN tests, lactose broth (Difco) served as the presumptive medium and brilliant green bile broth (Difco) and EC broth (Difco) as the confirmatory media for total and fecal coliforms, respectively. The media used in the MF tests were M-Endo agar (Difco) for total coliforms and M-FC agar (Difco) for fecal coliforms. Membrane filters used were Gelman GN-6, lot #81347. Incubation of total coliform MPN's and MF's was carried out at 35.0±0.5°C, while fecal coliform MPN's and MF's were incubated at 44.5±0.2°C. All liquid samples were neutralized with sodium thiosulfate prior to inoculation to destroy residual ozone.

Ozone Generator and Contactor

The ozone generator was obtained from Computerized Pollution Abatement Corporation (CPAC). It was an air-cooled, plate-type ozonator capable of producing 4 kg of ozone per day from air. Ozone was produced from air after the air had been dried to a dew point below -40°C.

The contactor (Figure 1) consisted of three aluminum columns, each 3.7 m high and 30 cm in diameter, connected in series and staggered vertically. Wastewater effluent* was pumped to the top of the first column at a flow rate of 75 l/min. The liquid emerged at the bottom of the column and flowed by gravity through a pipe to the top of the next column. This flow configuration was repeated to the third column.

The ozone-air gas mixture was injected at a rate of 75 l/min (25 l/min per column) through ceramic diffusers at the bottom of each column. The concentration of ozone in the gas was 9.0 mg/l gas. Since the gas stream was split equally among three columns, the liquid in each column received an applied ozone dosage of 3 mg/l. Thus, by the time the liquid emerged from the third column, a total, cumulative dosage of 9.0 mg/l had been applied. Residence time in each column was approximately 3 minutes.

With the above arrangement and the system operating continuously, we were able to sample any of the three columns at any desired time without the need to make adjustments in flow rates or ozone concentrations.

Analytical Method

Ozone was measured in the gas streams before and after each column by passing a sample gas stream through gas-washing bottles containing a 2% potassium iodide (KI) solution and then through a wet test meter to measure the volume. The liberated iodine was then titrated with a standardized solution of 0.0500 N sodium-thiosulfate. (2)

Experimental Design

A factorial experiment in randomized block design was performed to compare the two coliform enumeration methods as a function of applied ozone dosage. The three dosages chosen (i.e., 3, 6, and 9 mg/l) were considered fixed variables, since they were chosen to represent low, intermediate, and high dosage levels, corresponding to high, intermediate, and low coliform numbers, respectively, in the treated effluent. The order in which samples were collected at given dosage levels was randomized each day. Samples were collected at all three dosage levels on any given day, and thus a day was considered a replicate block experiment. A total of six replicates were performed.

When a liquid sample was collected (according to the predetermined randomized dosage order) it was immediately split into 8 equal fractions, labeled 1 through 8. Each label corresponded to either an MPN or an MF, as shown in Table 1. Two duplicate MPN's and two duplicate MF's were performed for each coliform type. Two technicians were utilized to inoculate the samples. One was assigned exclusively to MPN inoculations, while the other was assigned to MF's. Since it was obviously impossible to inoculate all eight sample fractions simultaneously, the order in which the sample fractions were inoculated was randomized to eliminate error due to standing. All inoculations were completed within one hour following sample collection.

Following completion of the six replicate experiments, separate analyses of variance (ANOVA) of the data for total coliforms and fecal coliforms were

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*From the Robert A Taft Laboratory Conventional Activated Sludge Pilot Plant

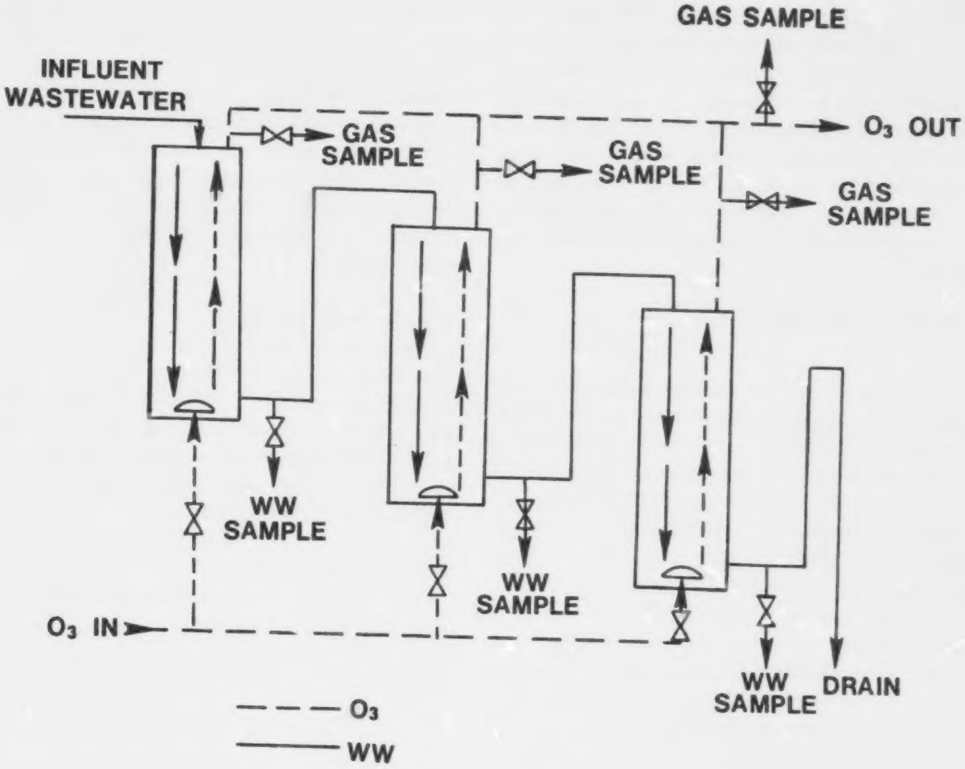

FIGURE 1. BUBBLE DIFFUSER OZONE CONTACTOR

performed in an IBM 370 computer using Biomedical Computer Programs BMD02V (Analysis of Variance for Factorial Design)(1). In each ANOVA there were three "main effects" under consideration and four "interaction effects" (Table 2). The Methods Main Effect (M) tested the overall difference in coliform numbers between methods. The Dosage Main Effect (D) tested the overall effect of ozone dosage on coliform numbers as determined by both methods combined. The Replication Main Effect (R) tested the overall effect of days (i.e., experimental replication) on coliform numbers. The Method-Dose Interaction Effect tested whether the difference between methods was consistent over all dosage levels or changed when different dosage levels were applied. The Method-Replication Interaction Effect tested whether the difference between methods was consistent or changed from day to day. The Replication-Dosage Interaction effect tested whether the difference between coliform numbers with respect to dosages was consistent or changed from day to day. The Replication-Method-Dose Interaction effect tested whether the difference between the method-dose interaction was consistent or changed from day to day.

TABLE 1. LABELING OF SAMPLE FRACTIONS ACCORDING TO COLIFORM ENUMERATION METHOD

Label Number	Analysis*
1	MPN - TC
2	MPN - TC
3	MF - TC
4	MF - TC
5	MPN - FC
6	MPN - FC
7	MF - FC
8	MF - FC

* TC = total coliforms; FC = fecal coliforms.

TABLE 2. EFFECTS TO BE TESTED BY ANALYSIS OF VARIANCE

1. Overall difference between methods (M)
2. Effect of dosage (D) on coliform numbers
3. Effect of experimental replication (R) on coliform numbers
4. Interaction between methods and doses (MD)
5. Interaction between methods and replicates (MR)
6. Interaction between replicates and doses (RD)
7. Interaction between replicates, methods, and doses (RMD)

RESULTS

Total Coliforms

Table 3 summarizes the differences between the MF and MPN methods of enumerating total coliform bacteria in ozonated effluent as a function of applied ozone dosage. Each number in the MF and MPN columns is the mean of 12 determinations (averaged over the 6 days). In Table 4, the same data are presented to illustrate the differences between the methods as a function of experiment days. Each number in the MF and MPN columns is the mean of 6 determinations (averaged over the 3 dosage levels). The ANOVA is presented in Table 5.

The analysis of variance indicated that the replication main effect (R) was highly significant ($p<0.01$). Thus, mean total coliform numbers, determined by both enumeration methods over all dosage levels, were significantly different from day to day (note row means in Table 4). This finding is not unexpected, since effluent quality varies widely from one day to another.

The methods main effect (M) was not significant (Table 5). Thus, the average number of total coliforms enumerated by each method over all dosage levels was similar. The method-replication interaction (MR) was also not significant. Thus, although total coliform numbers varied significantly between replicate runs (R effect), the average difference between the MF and MPN was not significant from one run to another.

The dosage main effect (D) was highly significant ($p<0.01$), indicating, as expected, that applied ozone dosage affected the final number of total coliforms as measured by both methods (note row means in Table 3). The difference between methods with respect to individual dosage levels (method-dosage interaction) was significant ($p<0.05$). Thus, although the two methods gave equivalent results when averaged over all dosage levels, differences did exist at one or more individual dosage levels. To determine where the differences occurred, the data were re-examined statistically at each dosage level. It was found that the MF gave significantly higher results than the MPN at the 6.0 mg/l dosage level, but the two methods produced similar results at the 3.0 and 9.0 mg/l dosage levels. From a practical microbiological viewpoint, the difference noted at the 6.0 mg/l dosage level was not of sufficient magnitude to warrant a strict interpretation of the data. Therefore, it is concluded that either the MPN or the MF technique can be used to enumerate total coliforms in ozonated secondary effluent.

The replication-dosage interaction (RD) was also highly significant, indicating that different effluent total coliform numbers resulted from the same given ozone dosage level on different days. This was probably due to the variable effluent quality on different days causing differences in ozone utilization efficiency,

TABLE 3. DIFFERENCE BETWEEN TOTAL COLIFORM METHODS AT EACH OZONE DOSAGE LEVEL

	Log_{10} Total Coliforms/100 ml (Standard Deviation)		
Dosage, mg/l	**MF**	**MPN**	**Both Methods**
3.0	5.13 (0.63)	5.31 (0.93)	5.22 (0.78)
6.0	3.97 (0.27)	3.58 (0.32)	3.78 (0.35)
9.0	3.33 (0.22)	3.15 (0.20)	3.24 (0.22)
All Dosages	4.14 (0.86)	4.01 (1.10)	

TABLE 4. DIFFERENCE BETWEEN TOTAL COLIFORM METHODS IN EACH REPLICATE RUN

	Log_{10} Total Coliforms/100 ml (Standard Deviation)		
Replicate	**MF**	**MPN**	**Both Methods**
1	3.62 (0.61)	3.29 (0.32)	3.45 (0.50)
2	3.83 (0.65)	3.92 (0.77)	3.88 (0.68)
3	4.32 (0.86)	4.26 (1.25)	4.29 (1.03)
4	4.45 (0.90)	4.31 (1.39)	4.38 (1.12)
5	4.06 (0.66)	3.84 (1.04)	3.95 (0.84)
6	4.59 (1.23)	4.47 (1.44)	4.53 (1.28)
All Replicates	4.14 (0.86)	4.01 (1.10)	

which in turn affected the final effluent coliform density. The 3-way interaction between replicates, methods, and dosages was also significant, indicating that the method-dose interaction was different from one day to another. Microbiologically, this effect is not considered important relative to the conclusions made above and will not be discussed in this paper.

TABLE 5. ANOVA FOR LOG TOTAL COLIFORM MPN, MF COMPARISON DATA

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Replicates (R)	5	9.46	1.89	76.06**
Methods (M)	1	0.30	0.30	4.90
RM	5	0.31	0.06	2.48
Doses (D)	2	50.20	25.10	49.10**
MD	2	0.96	0.48	4.59*
RD	10	5.11	0.51	20.54**
RMD	10	1.05	0.10	4.23**
Error	36	0.90	0.02	
Total	71			

* $p<0.05$
**$p<0.01$

Fecal Coliforms

Table 6 summarizes the differences between the MF and MPN methods of enumerating fecal coliforms in ozonated effluent as a function of applied ozone dosage. Each number in the MF and MPN columns is the mean of 12 determinations (averaged over the 6 days). In Table 7 the same data are presented to illustrate the differences between the methods as a function of experiment days. Each number in the MF and MPN columns is the mean of 6 determinations (averaged over the 3 dosage levels). The ANOVA is presented in Table 8.

TABLE 6. DIFFERENCE BETWEEN FECAL COLIFORM METHODS AT EACH OZONE DOSAGE LEVEL

Dosage, mg/l	$\log_{10}$ Fecal Coliforms/100 ml (Standard Deviation)		
	MF	MPN	Both Methods
3.0	4.12 (0.79)	4.51 (0.97)	4.32 (0.89)
6.0	2.87 (0.26)	2.77 (0.56)	2.82 (0.43)
9.0	2.27 (0.13)	2.27 (0.29)	2.27 (0.22)
All Doses	3.09 (0.91)	3.18 (1.17)	

The ANOVA indicated that R was highly significant ($p<0.01$). Thus, mean fecal coliform numbers determined by both methods over all dosage levels were significantly different from day to day (note row means in Table 7). This again reflects a variable effluent quality.

TABLE 7. DIFFERENCE BETWEEN FECAL COLIFORM METHODS IN EACH REPLICATE RUN

Replicate	$\log_{10}$ Fecal Coliforms/100 ml (Standard Deviation)		
	MF	MPN	Both Methods
1	2.49 (0.42)	2.48 (0.74)	2.48 (0.57)
2	2.84 (0.55)	2.81 (0.70)	2.82 (0.60)
3	3.35 (0.96)	3.56 (1.28)	3.45 (1.08)
4	3.20 (1.13)	3.38 (1.49)	3.29 (1.27)
5	3.09 (0.75)	3.05 (1.08)	3.07 (0.89)
6	3.55 (1.32)	3.83 (1.42)	3.68 (1.32)
All Replicates	3.09 (0.91)	3.18 (1.17)	

TABLE 8. ANOVA FOR LOG FECAL COLIFORM MPN, MF COMPARISON DATA

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Replicates (R)	5	11.44	2.29	32.71**
Methods (M)	1	0.18	0.18	3.00
RM	5	0.30	0.06	0.86
Doses (D)	2	53.90	26.95	34.55**
MD	2	0.80	0.40	5.71*
RD	10	7.85	0.78	11.14**
RMD	10	0.42	0.04	0.57
Error	36	2.43	0.07	
Total	71			

* $p<0.05$
**$p<0.01$

The methods main effect (M) was not significant (Table 8). Thus, the average number of fecal coliforms enumerated by each method over all dosage levels was similar. The MR interaction was also not significant. Thus, although the fecal coliform numbers varied significantly between replicate runs, the average difference between the MF and MPN was not significant from one run to another.

The dosage main effect (D) was highly significant ($p<0.01$), indicating again that applied ozone dosage affected the final numbers of fecal coliforms as measured by both methods (note row means in Table 6). The MD interaction was significant, indicating that differences between methods did exist at one or more individual dosage levels. To determine where the differences occurred, the data were re-examined satistically at each dosage level. The only significant difference found was at the 3.0 mg/l dosage level. At this level the MPN gave the higher result. From Table 6 it is clear that at a dosage of 3.0 mg/l the difference between the two methods was 0.39 log units. Although statistically significant, the difference was too minor to warrant a choice of one method over another. Therefore, it is concluded that either method will give equivalent results when quantifying fecal coliforms in ozonated secondary effluent.

The RD interaction was also highly significant ($p<0.01$), indicating that different effluent fecal coliform numbers resulted from the same given ozone dosage level on different days. Finally, the 3-way interaction between replicates, methods, and dosages was not significant.

CONCLUSIONS

Results from this experiment demonstrated that no significant difference exists between the MF and MPN techniques for enumerating total and fecal coliform bacteria in secondary effluent disinfected with ozone. Ozone dosage significantly affected total and fecal coliform numbers in the treated secondary effluent. Total and fecal coliform densities following ozonation varied significantly between replicate experiments, indicating effluent quality may have affected the variation in the final effluent numbers.

The difference between methods was not consistent from one dosage level to another, indicating that significant differences did exist between methods at one or more dose levels. However, the magnitude of the difference was not considered substantial enough to warrant a choice of one method over another. Finally, the differences between the MPN and MF techniques were not affected by day-to-day changes in effluent quality. From the foregoing analysis, it is reasonable to conclude that either the MPN or the MF technique can be used to enumerate total and fecal coliform bacteria in secondary effluent that has been disinfected with ozone. However, since the Code of Federal Regulations specifies the MPN as the method of choice in enforcement situations, compliance with the US EPA Environmental Monitoring and Support Laboratory's (EMSL) equivalency testing program would still be required to get approval to use another method such as the MF.

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DISCUSSION

DR. RICE, *Jacobs Engineering*: Mark, I did not look at your diagram hard enough at the beginning. Did you have any ozone in the off gases, and, if you did, then should we be talking ozone dosage, or should we be talking ozone utilized?

MR. MECKES: We did do off gas analysis on these experiments. I did correlate them, and at this point in time I would rather defer the results from that to Al's talk who will be discussing the relationship between absorbed ozone and coliform reduction.

DR. RICE: I am not being a nitpicker. I am only concerned with the final engineering design of the plant. It probably should be made on the basis of the amount of ozone you are going to use rather than the amount of ozone you are actually dosing, so that was the point.

MR. AIETA, *Stanford University*: Mark, in your statistical analysis did you take a look at the relative precisions of the MPN and the MF methods?

MR. MECKES: I certainly did, Marco, and we found that for this wastewater there was no difference between the precisions.

MR. AIETA: As measured by the F test?

MR. MECKES: Measured by the T-test.

MR. WHITE: I have a practical question. I cannot for the life of me understand why we are worrying about the membrane filter technique versus the MPN when we do not know anything about what the effects are of the coliform organisms in the effluent insofar as disease is concerned, and I would just like to implant that idea here, because I think we are getting far away from the practical application of disinfection. You do not have to answer that question. I do not think there is an answer, but I would just like to point it out.

MR. VENOSA: How do you measure disinfection, George?

MR. WHITE I have no way of measuring disinfection except going through all the rigamarole that the State of California did beginning in 1920 to try to find out what did represent a threat to public health and what did not represent threat to public health. If you read what I write, if you read what other people write, they claim that in surface water, if the total coliform count was in excess of 1,000 per 100 mililiter, this was the factor they used to establish that it was pollution and there should be something done about it. But the only work I know is Stevenson's, who did some work on the threat of disease and how it affected public health. So I cannot understand why we are wasting time on the membrane filter technique versus the MPN because it is so imprecise.

MR. VENOSA: The membrane filter technique is much more precise that the MPN (except in the ozonated effluent study we found that they were similar). The only reason we compared the two methods was basically because of our ozone contactor study. We did not want to use MPN's because it takes four days to get a result, while it only takes twenty-four hours to get a result with the MF. We wanted, first of all, to see if there were any differences between the two methods so that then we could use the MF method and do more experiments in a shorter amount of time, and spend fewer research dollars to determine the differences between ozone contactors. The only reason we are doing that is to establish a methodology for us to use. We all agree that there are limitations to coliforms, and what their significance is in relation to disease transmission. We are merely using them as we always have in sanitary engineering to indicate the possible presence of pathogens, and we cannot measure pathogens. There are no precise methods of measuring pathogens in the environment. We have to rely on the fecal coliforms and the total coliforms to indicate the presence of those pathogens, and that is why we are doing it. The only way that we can reliably measure disinfection efficiency today is by measuring the coliform numbers.

MR. WHITE: Well, you have answered the question. The membrane filter technique gets the answer fast since it is imprecise insofar as coliform numbers versus public health. Then your membrane filter technique is the answer. It does not take so long.

DR. REYNOLDS, *Utah State Univ*: In response to George's question, as Al knows, we ran comparisons of MPN and MF tests in the lagoon study that I reported on yesterday morning, and we did find a substantial difference between coliforms enumerated by MPN and MF. It was not as sound a statistical test as was just described here, that is, it was not a randomized block statistical test, but it was splitting samples, identical samples, and running them with various technicians who happened to be there in connection with the project (who were competent by the way), and we did see a significant difference between those two methods. It may be characteristic of the lagoon effluent we were working with as compared to an ozonated effluent, but we found differences. I think it is important that this work has been done to substantiate the validity of the research that is going to follow in the next paper.

MISS ELLIOTT: I was just curious if you were using an enrichment broth prior to the full incubation. If you did, how significant was that effect?

MR. MECKES: We did not bother to consider that, especially since the results from this study showed that we did not have any significant difference by just using the one step technique, so therefore we found no enhancement through enrichment would be necessary anyway. Why go through the additional effort when one does not have to?

18.

COMPARATIVE EFFICIENCIES OF OZONE UTILIZATION AND MICROORGANISM REDUCTION IN DIFFERENT OZONE CONTACTORS

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INTRODUCTION

This project was initiated to evaluate various types of ozone contacting devices operating in parallel on the same source of secondary effluent, so that the disinfection efficiency of ozone can be optimized and operating costs defined and quantified. The types of contactors being studied in this project were selected to be representative of various generic devices commericially available or in widespread use in the chemical engineering field. It was the intent of the project to compare different contactor types on the basis of log total and fecal coliform reduction and ozone utilization under controlled conditions.

At the International Ozone Institute's *Symposium on Advanced Ozone Technology* held in Toronto, Ontario, Canada on November 16-18, 1977, Scaccia and Rosen stated: ". . . properly designed ozone contactors show similar performance such that there is no 'best' state-of-the-art ozone contactor." (7) They went on to say that any generic type contactor can be optimally designed to achieve the same degree of mass transfer, ozone disinfection, and ozone utilization at approximately the same *applied ozone dose.* The selection of a contactor then becomes one primarily of equipment economics, which are affected by contact time, materials, and energy trade-offs.

This paper will present evidence that there *can* be a "best state-of-the-art ozone contactor" when one considers and understands all factors responsible for the operating characteristics of the specific contactors. For purposes of understanding this and subsequent discussions, it is necessary to start off with some fundamental definitions of terms.

1. Applied Ozone Dose (D)

$$D = Y_1(Q_G/Q_L) \qquad (1)$$

where Y_1 = concentration of ozone in the carrier gas, mg $O_3/1_{gas}$

Q_G = carrier gas flow rate, 1_{gas}/min

Q_L = liquid flow rate, 1_{liq}/min

By inspection of equation 1, it is clear that the applied dose can be varied either by changing the ozone concentration in the inlet carrier gas (Y_1) or by changing the Q_G/Q_L ratio.

2. Percent Ozone Utilization (%U)

$$\%U = \frac{Y_1 - Y_2}{Y_1}(100) \qquad (2)$$

where Y_2 = concentration of ozone in the gas leaving the contactor, mg $O_3/1_{gas}$.

3. Actual Ozone Utilization (U) or Absorbed Dose

$$U = D \times \text{Fraction Utilized}$$

$$= Y_1(Q_G/Q_L)\frac{(Y_1 - Y_2)}{Y_1}$$

$$= Q_G/Q_L\ (Y_1 - Y_2) \qquad (3)$$

In this phase of our investigation, three contactors were studied: 1. a packed column (PC); 2. a positive pressure injector (PPI); and 3. a bubble diffuser (BD). The above contactors were all designed and fabricated in-house, with the mixing head for the PPI provided by the Union Carbide Corporation.

MATERIALS AND METHODS

Source of Wastewater

Secondary effluent was obtained from the conventional activated sludge pilot plant at the Robert A. Taft Laboratory, Cincinnati, Ohio. Wastewater entering the plant was of municipal origin. Effluent from the final clarifer was filtered through a multi-media pressure filter (Baker Filtration Company, Model HRC-3OD) and then split equally between two of the contactors for parallel evaluation. Total flow rate was 150 l/min (75 1/min per contactor).

It was found in an earlier phase of the project (8) that unusually high pressure drops occurred over the length of the packed column contactor when effluent was pumped through it. The pressure drops were high enough (greater than 75 cm H_2O) to cause flooding of the column. To rectify this situation, a small amount of antifoam material (Dow-Corning Antifoam emulsion H-10) was added to the filtered secondary effluent ahead of the packed column. Only this contactor received antifoam. This eliminated foaming, which was causing the liquid hold-up in the packed column, and the pressure drops returned to normal (less than 20 cm H_2O).

Ozone Concentration

Ozone was generated from air in a plate-type, corona discharge ozonator (Computerized Pollution Abatement Corporation Model OZ-180-G). Gas from the generator was split into two lines, each with a rotameter and valve for flow control. These lines served as ozone feed to the contactors.

The carrier air was filtered through a prefilter (Pall Trinity Micro Corporation Model No. MCC1001SV160) and then dried to a dew point of less than −40°C by an activated alumina dryer (Pall Trinity Micro Corporation 25 HAl). The dew point was monitored by a Shaw Mini-Hygrometer, which was calibrated against an Alnor Dew Pointer (Model 7300, Alnor Instrument Co., Niles, Ill.). Carrier air leaving the drying unit was filtered through an afterfilter (Pall Trinity Micro Corporation Model No. MC-Cl10lEC12) prior to entering the ozonator.

Ozone Contactors

Figure 1 is a schematic diagram of the packed column contactor. This was a 230 mm diameter glass column packed with 3.1m of 12mm ceramic intalox saddles. A teflon redistributor plate was located midway in the column. Filtered secondary effluent entered the top of the column and exited at the bottom. Mean residence time of the secondary effluent was approximately 35 seconds (measured by a dye study) at a liquid flow rate of 75 1/min and a gas flow rate of 40 1/min. Ozone, injected at the bottom of the column, flowed upward countercurrent to the secondary effluent and exited at the top. A gas sample tap was located on the exhaust line. Effluent from the PC was directed to two 200-liter covered hold tanks connected in series to allow an additional five minutes contact time with the ozone residual. Liquid sample taps were located as shown in Figure 1.

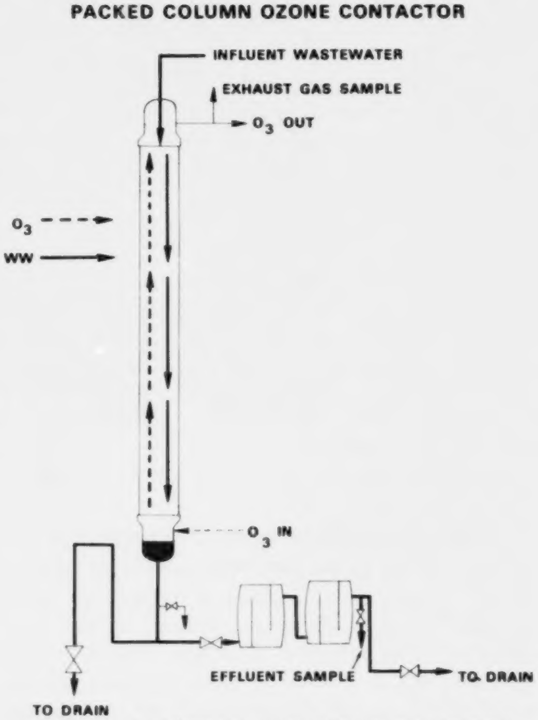

FIGURE 1. PACKED COLUMN OZONE CONTACTOR

Figure 2 is a schematic diagram of the PPI contactor. Ozone enriched gas was injected into the influent liquid flow in a specially designed "tee-mixing chamber" under a slight positive

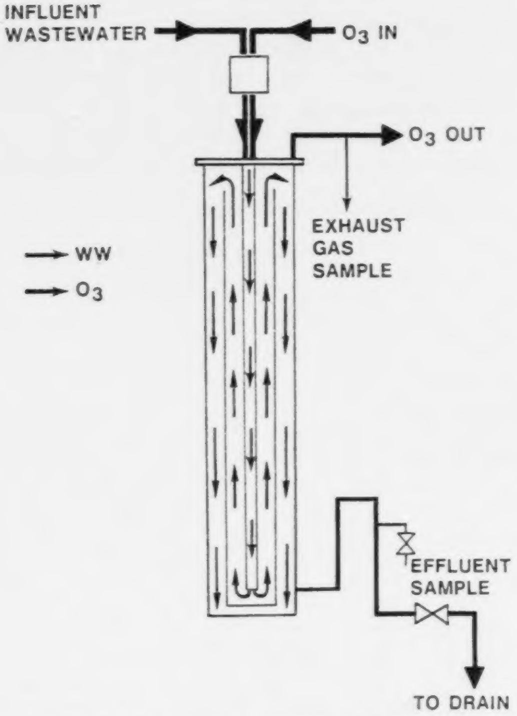

FIGURE 2. PRESSURE INJECTOR CONTACTOR

pressure of 50 to 60 kPa. The gas-liquid mixture flowed concurrently downward through an inner cylinder. Upon reaching the bottom of the inner cylinder, the mixture reversed direction and flowed upwards through a larger cylinder. At the top, the gas and liquid separated, the gas flowing to the exhaust gas line and the liquid flowing downwards through a second concentric cylinder, finally emerging at the bottom to a drain line. Mean liquid residence time in the system was approximately 15 seconds (measured by a dye study) at a liquid flow rate of 75 l/min and a gas flow rate of 40 l/min.

Figure 3 is a schematic diagram of the bubble diffuser (BD) contactor. It consisted of three aluminum columns, each 3.7m high and 300mm in diameter, connected in series by PVC piping. The three columns were staggered vertically so that secondary effluent, which has been pumped to the top of the first column, could flow by gravity to each of the two successive columns. The ozone enriched gas was injected through domed ceramic diffusers (Norton Chemical Process Products Division) located at the bottom of each column and connected in parallel to a common header pipe. The first column received 50% of the total gas flow, while the other two columns received equal fractions of the remaining 50%. Gas and liquid sample taps were located as shown in Figure 3. Since the BD was operated as a system, treated effluent samples were collected at the sample tap of the last column only, while exhaust gas samples were collected after the exhaust gas from each column had been combined into a common line (see Figure 3). Mean liquid residence time in the BD contactor, measured by a dye study was 9.4 minutes at a liquid flow rate of 75 l/min and a gas flow rate of 40 l/min.

Sampling

All effluent samples were grab samples and were analyzed for total and soluble chemical oxygen demand (TCOD and SCOD, respectively). total organic carbon (TOC), total Kjeldahl nitrogen (TKN), ammonia-nitrogen ($NH_4^+ - N$), nitrate-nitrogen ($NO_3^- - N$), nitrite-nitrogen ($NO_2^- - N$), total suspended solids (TSS), turbidity (turb), temperature, and pH by Standard Methods (1) or U.S. EPA approved methods (6). Ozone concentration in the inlet and exhaust gas was determined iodometrically by the method of Birdsall, Jenkins, and Spadinger (3), and a wet test meter was used to measure gas volumes.

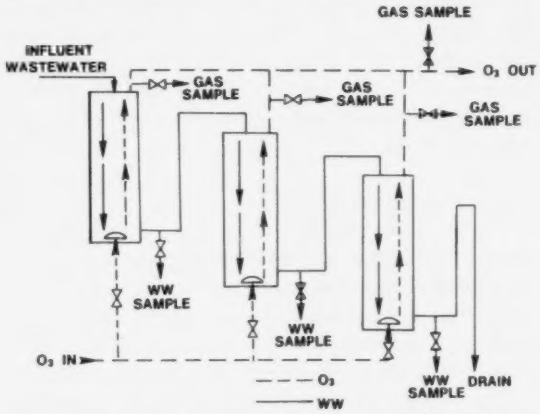

FIGURE 3. BUBBLE DIFFUSER OZONE CONTACTOR

Samples collected for bacteriological analysis were assayed for total and fecal coliforms by the standard Membrane Filtration (MF) method (1), using 0.45 μm GN-6 membrane filters (Gelman Instrument Company). Media for total and fecal coliforms were M-Endo Agar (Difco) and M-FC

Agar (Difco), respectively. Plates were incubated in water baths adjusted to 35 ± 0.5 °C for total coliforms and 44.5 ± 0.2 °C for fecal coliforms.

Colonies of total coliforms were randomly picked and inoculated into Lauryl Sulfate Tryptose broth (LST) (Difco) in fermentation tubes and incubated for 24 to 48 hours at 35 ± 0.5 °C for confirmation. Positive confirmation was indicated by the production of gas in the fermentation tubes. Fecal coliform colonies were randomly picked, inoculated into LST medium, and incubated for 24 to 48 hours at 35 °C. A loopful of suspension from gas-positive tubes was transferred to EC broth (Difco) in fermentation tubes and incubated for 24 hours at 44.5 ± 0.2 °C. Positive confirmation was indicated by production of gas in the EC medium at the end of the incubation period.

During this study a total of 561 total coliform colonies were picked from the membrane filters used on effluent and ozonated effluent. Of these, 488 were confirmed to be total coliforms, resulting in a verification percentage of 87%. The percent verification of fecal coliforms was 95% (538 confirmed out of 566 isolates picked).

Experimental Design

Testing of the different contactor types was performed using a split plot design where the whole plots were arranged in a balanced incomplete block design. A discussion of this type of design can be found in Federer (4). According to the design, two contactors per day were set up in parallel. Secondary effluent was pumped to each ozone contactor at a rate of 75 l/min. The ozone-air mixture was fed to the contactors at various gas flow rates and ozone concentrations. Three different pre-selected dose levels were used.

The three contactors formed the whole plot portion of the design. The whole plots were arranged in a balanced incomplete block design due to the physical restriction that only two of the contactors could be set up in parallel at any given time. Thus, two contactors per day were tested with both contactors receiving the same applied dose at any given time. All three dose levels were used each day. The order of the dose levels was balanced so that each dose level was used first, second, and third in a day the same number of times for each contactor. This was done to minimize the effect of any trend in the secondary effluent over the day. Randomization consisted of randomizing the order in which the pairs of contactors were compared.

Thus, an entire experiment (block), encompassing two contactors and all three dose levels, was conducted in one day. The dependent varibles (i.e., performance criteria) forming the basis of comparison of the contactors were total and fecal coliform log reduction ($\log_{10} N_o/N$, where N_o = initial coliform number, and N = final coliform number), ozone utilization, and percent ozone utilization. Separate analyses of variance (ANOVA) were performed to compare the contactors on the basis of each of the four performance criteria. The typical form of each ANOVA was as follows:

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8			
Contactors (C_j) (adj. for blocks)	2			
Whole plot error (B_iC_j)	7			
Split Plot Analysis				
Doses (D_k)	2			
C_jD_k	4			
B_iD_k	16			
$B_iC_jD_k$	14			
Total	53			

The contactor main effect tested the overall difference in performance in the contactors. Its F-value was computed using the mean square for contactors ratioed against the mean square for the whole plot error. A significant contactor effect led to a comparison of the contactor means adjusted for blocks using Tukey's HSD multiple comparison procedure (5). The dose main effect tested the effect of different ozone dose levels on the combined performance of the contactors. The dose F-value was the ratio of the dose mean square to the block-by-dose mean square. The contactor-by-dose interaction tested whether the effect of the contactors was the same or different for each dose level. The contactor-by-dose interacton F-test was the ratio of the

C_jD_k mean square to the $B_iC_jD_k$ mean square. A significant contactor-by-dose interaction meant that the effect of the contactors had to be evaluated separately for each dose level. Finally the block-by-dose interaction was tested using the B_iD_k mean square divided by the $B_iC_jD_k$ mean square. This tested whether the effect of dose levels was consistent or changed from day to day.

Data analyses were facilitated by an IBM computer, using the Statistical Analysis System (SAS) General Linear Model (GLM) procedure (2).

RESULTS AND DISCUSSION

Effect of Increasing Q_G/Q_L at a Constant Y_1

The objective of the first balanced incomplete block experiment was to evaluate the effect of increasing the applied ozone dose to each contactor by increasing the Q_G/Q_L ratio while maintaining a constant inlet ozone concentration (Y_1). Secondary effluent flow rate (Q_L) to each contactor was 75 l/min and the gas flow rates (Q_G) were 25, 50, and 75 l/min, corresponding to Q_G/Q_L ratios of 0.33, 0.67, and 1.00, respectively. The concentration of ozone in the inlet gas was maintained at 10.0 mg/l_{gas} (approximately 0.8 weight percent). Thus, the three applied ozone dose levels studied were 3.3, 6.7, and 10.0 mg/l. This experiment was conducted during the period of June 22 to July 21, 1978.

Quality of Wastewater

In Table 1, the minimum, mean, and maximum values of the physical-chemical and bacteriological characteristics of the filtered pilot plant effluent prior to ozonation are presented. Quality was typical of a well-treated secondary effluent. Note that the total coliform density was approximately four times the fecal coliform density.

TABLE 1. SUMMARY OF FILTERED PILOT PLANT EFFLUENT CHARACTERISTICS PRIOR TO OZONATION (JUNE 22 to JULY 21, 1978)

Parameter	Mean	Minimum	Maximum
TCOD, mg/l	35	22	46
SCOD, mg/l	31	22	38
TOC, mg/l	11.5	7.7	16.5
TSS, mg/l	3.8	1.4	7.2
NH_4^+-N, mg/l	13.4	7.7	19.2
TKN, mg/l	17.1	10.0	28.7
Turbidity, JTU	2.2	0.6	3.8
pH	—	7.2	8.3
Temperature, °C	22	20	23
Total Coliforms/ 100 ml*	2.6×10^6 (geometric mean)	2.4×10^5	7.9×10^6
Fecal Coliforms/ 100 ml**	6.5×10^5 (geometric mean)	1.4×10^5	2.0×10^6

* Adjusted for 87% colony verification.
**Adjusted for 95% colony verification.

Percent Ozone Utilization

In Table 2, the percent ozone utilization data for the first experiment are summarized. In Table 3, the analysis of variance (ANOVA) for those data is presented. In the whole plot analysis, the ANOVA revealed that the Block Main Effect (B_i) was not significant, indicating that very little day to day variation in percent ozone utilization occurred in the three contactors. The Contactor Main Effect (C_j) was highly significant ($p < 0.01$). This indicated that the mean overall percent ozone utilization (i.e., averaged over all 3 dose levels) was significantly different in at least two of the three contactors (note column means in Table 2). To determine where the significant differences occurred, Tukey's HSD multiple comparison test (5) was used. Results indicated that overall percent ozone utilization was highest in the bubble diffuser, followed by the packed column, and then the positive pressure injector. The differences were significant between all three contactors ($p < 0.05$).

TABLE 2. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSAGE (INCREASING Q_G/Q_L, CONSTANT Y_1)

	Mean Percent Ozone Utilization (Standard Deviation)			
Dose, mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean % U, All Contactors
3.3	77 (2)	85 (4)	90 (2)	84 (6)
6.7	53 (3)	58 (2)	86 (2)	66 (15)
10.0	41 (3)	46 (3)	81 (3)	56 (18)
Mean % U, All Doses	57 (16)	63 (17)	86 (5)	

In the split plot analysis, the ANOVA revealed that the Dose Main Effect (D_k) was highly significant ($p < 0.01$). This indicated that the percent ozone utilization in all contactors combined was significantly different at each dose level studied (note row means in Table 2). As dose was increased, percent ozone utilization in all contactors decreased. The Contactor-Dose Interaction (C_jD_k) was also highly significant ($p < 0.01$, Table 3), indicating that the differences in percent ozone utilization between contactors were not the same at each dose level. To facilitate visualization of these effects, the data from Table 2 are depicted graphically in Figure 4. It is clear that the rate of decrease in percent ozone utilization, as a function of applied dose, was substantially less in the bubble diffuser than in either the packed column or the positive pressure injector. When the effects of the contactors were re-examined statistically at each individual dose level, it was found that the contactor main effect was significant even at the lowest dose level, and that the bubble diffuser significantly outperformed the packed column, which in turn outperformed the positive pressure injector at each dose level studied.

Finally, the Block-Dose Interaction (B_iD_k) was not significant (Table 3), indicating that the effect of dose levels on percent ozone utilization in all contactors was consistent from day to day. The paired percent ozone utilization data on each experiment day (Block) are plotted in Figure 5, to aid visualization of the performance differences between contactors and the performance consistency of each contactor from day to day. The consistency in the percent ozone utilization data was likely due to the relative lack of change in wastewater quality over the duration of the study period.

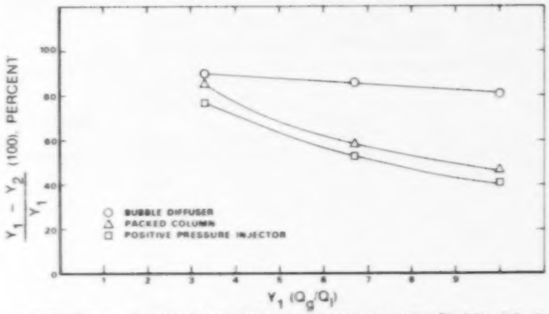

FIGURE 4. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE

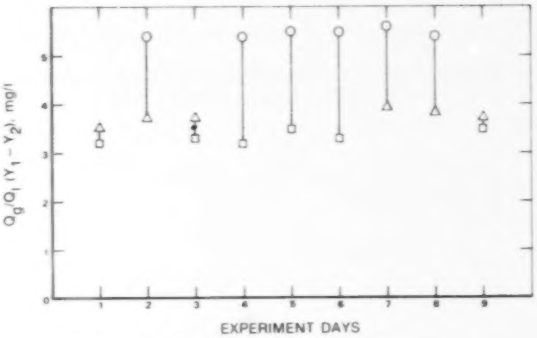

FIGURE 5. OVERALL MEAN PERCENT OZONE UTILIZATION PER REPLICATE RUN

TABLE 3. ANOVA FOR PERCENT OZONE UTILIZATION DATA
(INCREASING Q_G/Q_L, CONSTANT Y_1)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	63.12	7.89	2.20
Contactors (C_j) (adj. for blocks)	2	1911.44	955.72	267.01*
Whole Plot Error (B_iC_j)	7	25.06	3.58	
Split Plot Analysis				
Doses (D_k)	2	7433.33	3716.67	812.55*
C_jD_k	4	1694.56	423.64	169.81*
B_iD_k	16	73.19	4.57	1.83
$B_iC_jD_k$	14	34.93	2.49	
TOTAL	53			

*Significant at the $p < 0.01$ level

Actual Ozone Utilization (Absorbed Dose)

In Table 4 the actual ozone utilization data (i.e., the amount of ozone actually transferred to, but not necessarily consumed by, the wastewater) from the first experiment are summarized. The ANOVA for those data are presented in Table 5. In the whole plot analysis, the ANOVA revealed that B_i was not significant, again indicating very little change in overall ozone utilization in all contactors from day to day. C_j was highly significant ($p < 0.01$). Thus, the mean overall ozone utilization was significantly different in at least two of the three contactors (note column means in Table 4). Using Tukey's HSD multiple comparison test, actual overall ozone utilization was highest in the bubble diffuser, then the packed column, and finally the positive pressure injector.

TABLE 4. MEAN OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Q_G/Q_L, CONSTANT Y_1)

	Mean Ozone Utilization, mg/l (Standard Deviation)			
Dose, mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean U, mg/l All Contactors
3.3	2.5 (0.1)	2.8 (0.1)	3.0 (0.1)	2.8 (0.2)
6.7	3.5 (0.2)	3.8 (0.1)	5.7 (0.2)	4.4 (1.0)
10.0	4.0 (0.2)	4.5 (0.3)	7.9 (0.3)	5.2 (1.7)
Mean U, mg/l, All Doses	3.3 (0.6)	3.7 (0.8)	5.5 (2.1)	

TABLE 5. ANOVA FOR OZONE UTILIZATION DATA (INCREASING Q_G/Q_L, CONSTANT Y_1)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	0.20	0.024	1.52
Contactors (C_j) (adj. for blocks)	2	11.75	5.87	366.22**
Whole Plot Error ($B_i C_j$)	7	0.11	0.016	
Split Plot Analysis				
Doses (D_k)	2	65.72	32.86	1152.25**
$C_j D_k$	4	21.80	5.45	565.90**
$B_i D_k$	16	0.46	0.028	2.96*
$B_i C_j D_k$	14	0.13	0.0096	
TOTAL	53			

* significant at the $p<0.05$ level
**significant at the $p<0.01$ level

These differences were significant at the $p<0.05$ level.

In the split plot analysis, the ANOVA revealed that D_k was highly significant ($p< 0.01$). Thus, the amount of ozone transferred to the wastewater in all three contactors combined was significantly different at each dose level studied (note row means in Table 4). As dose was increased, ozone utilization in all contactors increased. The C_jD_k interaction was also highly significant ($p < 0.01$, Table 5), again indicating that differences in ozone utilization between contactors were not the same at each dose level. A graphical representation of these data is presented in Figure 6. It is clear that the rate of increase in ozone utilization as a function of applied dose was substantially higher in the bubble diffuser than in either of the other two contactors. When the effects of the contactors were re-examined statistically at each individual dose level, it was found that the differences were significant even at the lowest dose level. Ozone utilization at each dose level was highest in the bubble diffuser and lowest in the positive pressure injector.

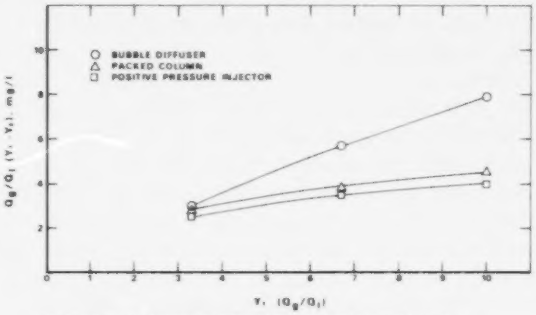

FIGURE 6. MEAN OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE

Finally, the B_iD_k interaction was significant at the $p< 0.05$ level (Table 5), indicating that the effect of dose levels on ozone utilization in all contactors was variable from day to day. This is not consistent with the findings from the percent ozone utilization data, and the reason for the discrepancy is not altogether clear. However, the result is not considered important enough to warrant further investigation. The paired ozone utilization data on each experiment day (block) are plotted in Figure 7 to aid visualization of the daily performance differences between contactors.

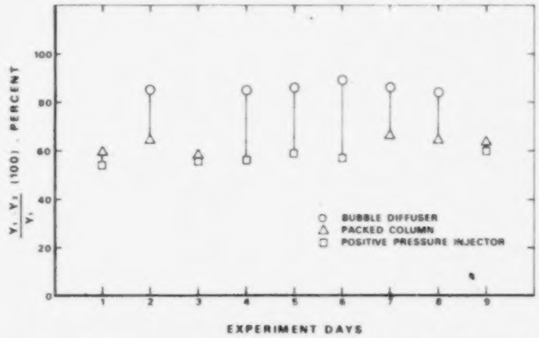

FIGURE 7. OVERALL MEAN OZONE UTILIZATION PER REPLICATE RUN

Total Coliform Log Reduction (TCLR)

In Table 6, the total coliform log reduction (TCLR) data from the first experiment are summarized. The ANOVA for those data are presented in Table 7. In the whole plot analysis, the only significant effect noted was C_j, indicating that mean TCLR was significantly different in at least two of the three contactors (note column means in Table 6). Tukey's HSD multiple comparison test of the data revealed that overall mean TCLR was higher in the bubble diffuser than in either the packed column or the positive pressure injector ($p < 0.05$). However, the overall mean TCLR in the latter two contactors was not significantly different. Thus, the significant C_j effect in Table 7 was due entirely to the higher TCLR in the bubble diffuser contactor. Furthermore, even though the actual ozone utilized in the packed column was significantly higher than in the pressure injector (Table 5), the increased utilization was not sufficient to cause a noticeable difference in total coliform reduction between the two contactors.

TABLE 6. MEAN TOTAL COLIFORM LOG REDUCTION (TCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Q_G/Q_L, CONSTANT Y_1)

	TCLR (Standard Deviation)			
Dose, mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean TCLR All Contactors
3.3	2.90 (0.36)	2.85 (0.36)	3.12 (0.10)	2.96 (0.31)
6.7	2.94 (0.44)	3.13 (0.10)	3.56 (0.25)	3.21 (0.38)
10.0	3.04 (0.34)	3.31 (0.11)	4.52 (0.55)	3.62 (0.75)
Mean TCLR, All Doses	2.96 (0.36)	3.10 (0.29)	3.73 (0.69)	

In the split plot analysis, the ANOVA revealed that both the D_k and the C_jD_k interaction were highly significant ($p < 0.01$). Thus, the applied ozone dose significantly affected the magnitude of the total coliform reduction in all contactors, and the differences in TCLR between contactors were not the same at each dose level. These effects are easily visualized by examining Figure 8. Note the apparent differences in slopes of the three TCLR curves. When the data were re-evaluated at each individual dose level, it was found that: 1. there was no significant difference in TCLR between the bubble diffuser, the packed column, or the positive pressure injector at applied ozone doses of 3.3 and 6.7 mg/l; 2. at an applied ozone dose of 10.0 mg/l, TCLR was significantly higher in the bubble diffuser than in the packed column, and higher in the packed column than in the positive pressure injector ($p < 0.05$). The geometric mean total coliform densities in the three contactors after 10.0 mg/l applied ozone dose were: 160/100 ml in the bubble diffuser, 1300/100 ml in the packed column, and 2900/100 ml in the positive pressure injector.

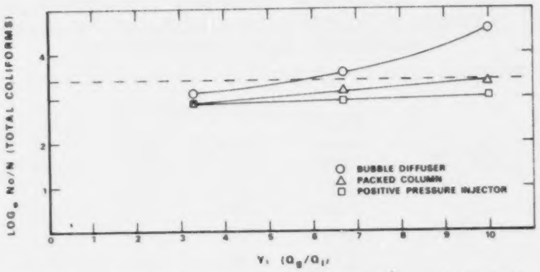

FIGURE 8. MEAN LOG TOTAL COLIFORM REDUCTION AS A FUNCTION OF APPLIED OZONE DOSE

TABLE 7. ANOVA FOR TOTAL COLIFORM LOG REDUCTION DATA (INCREASING Q_G/Q_L, CONSTANT Y_1)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	0.68	0.086	1.81
Contactors (C_j) (adj. for blocks)	2	1.13	0.57	12.00*
Whole Plot Error (B_iC_j)	7	0.33	0.047	
Split Plot Analysis				
Doses (D_k)	2	4.08	2.04	37.75*
C_jD_k	4	2.76	0.69	10.87*
B_iD_k	16	0.86	0.054	0.85
$B_iC_jD_k$	14	0.89	0.063	
TOTAL	53			

*significant at the $P<0.01$ level

Fecal Coliform Log Reduction (FCLR)

In Table 8, the fecal coliform log reduction (FCLR) data from the first experiment are summarized. The ANOVA for those data are presented in Table 9. In the whole plot analysis, the block main effect (B_i) was highly significant

TABLE 8. MEAN FECAL COLIFORM LOG REDUCTION (FCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Q_G/Q_L, CONSTANT Y_1)

	FCLR (Standard Deviation)			
Dose, mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean FCLR, All Contactors
3.3	2.99 (0.38)	3.08 (0.40)	3.22 (0.16)	3.10 (0.33)
6.7	3.08 (0.59)	3.16 (0.42)	3.71 (0.24)	3.32 (0.50)
10.0	3.26 (0.13)	3.34 (0.46)	4.29 (0.36)	3.63 (0.58)
Mean FCLR, All Doses	3.11 (0.40)	3.19 (0.42)	3.74 (0.51)	

TABLE 9. ANOVA FOR FECAL COLIFORM LOG REDUCTION DATA (INCREASING Q_G/Q_L, CONSTANT Y_1)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	1.47	0.18	13.23*
Contactors (C_j) (adj. for blocks)	2	0.60	0.30	21.76*
Whole Plot Error ($B_i C_j$)	7	0.097	0.014	
Split Plot Analysis				
Doses (D_k)	2	2.56	1.28	24.04*
$C_j D_k$	4	1.29	0.32	5.85*
$B_i D_k$	16	0.85	0.053	0.97
$B_i C_j D_k$	14	0.77	0.055	
TOTAL	53			

*significant at $p<0.01$ level.

($p < 0.01$). This is not consistent with the ANOVA's for the other three dependent variables (Tables 3, 5, and 7). On the first experiment day, when the packed column and positive pressure injector were run in parallel, the FCLR's in both contactors were much lower than on any other day. This observation was apparently of sufficient magnitude to cause the significant B_i effect. Noteworthy among the effluent quality characteristics on this particular day is the fact that the fecal coliform density prior to ozonation was approximately 10-fold lower than on any other day.

The C_j effect was also highly significant ($p < 0.01$), again indicating that mean fecal coliform reduction was different in at least two of the three contactors (note column means in Table 8). Application of Tukey's HSD multiple comparison test to the data revealed that, just as with the TCLR data, FCLR in the bubble diffuser was significantly higher than in either the packed column or the positive pressure injector. Moreover, there was no difference in overall FCLR between the latter two contactors.

In the split plot analysis, the ANOVA revealed that both the D_k and the C_jD_k interaction were highly significant ($p < 0.01$). Again the applied ozone dose significantly affected the magnitude of the fecal coliform reduction in all contactors, and the differences in FCLR between contactors were not the same at each dose level. These effects are graphically depicted in Figure 9. The slopes of the curves in Figure 9 appear different. Statistical re-evaluation of the data at each dose level disclosed that there was no significant difference in FCLR between the packed column and positive pressure injector at each dose level, whereas FCLR in the bubble diffuser was significantly higher than in the other two contactors at 6.7 and 10.0 mg/l, but not at 3.3 mg/l. The geometric mean fecal coliform densities in the three contactors after 10.0 mg/l applied ozone dose were: 60/100 ml in the bubble diffuser, 290/100 ml in the packed column, and 350/100 ml in the positive pressure injector. At 6.7 mg/l applied ozone dose, the geometric mean fecal coliform densities were 200/100 ml in the bubble diffuser, 320/100 ml in the packed column and 420/100 ml in the positive pressure injector.

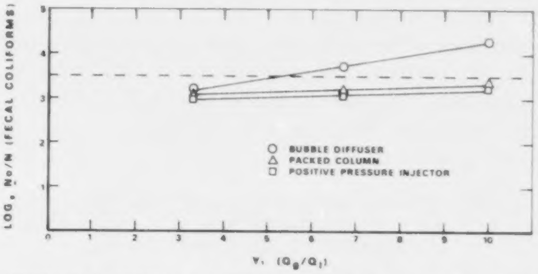

FIGURE 9. MEAN LOG TOTAL COLIFORM REDUCTION AS A FUNCTION OF APPLIED OZONE DOSE

Coliform Reduction vs. Ozone Utilization

Scaccia and Rosen concluded in their study that ". . . equivalent disinfection will be achieved at substantially the same utilized ozone dose, independent of contactor type studied."(7). To test this, we plotted the total and fecal coliform log reduction data in all contactors against the ozone utilization data in all contactors. Results are shown in Figures 10 and 11. Data from the three individual contactors are differentiated by the symbols, as defined in the legends. It is clear that our data are consistent with the conclusion reached by Scaccia and Rosen, namely, that equivalent total or fecal coliform reductions are achieveable in a given wastewater independent of contactor type as long as the absorbed dose is the same. However, we have shown in the foregoing data analyses and discussion that equivalent absorbed doses (i.e., utilization) necessary to achieve a given bacteriological standard may not be possible in some contactors. The dashed horizontal lines depicted on Figures 10 and 11 represent calculated total and fecal coliform log reductions, respectively, necessary to achieve 1000 total coliforms/100 ml and 200 fecal coliforms/100 ml in our filtered pilot plant effluent. The corresponding ozone utilization needed to achieve those bacteriological levels is approximately 4.8 mg/l. Referring back to Figure 6 wherein applied ozone dose is plotted against absorbed ozone dose (utilization) in each contactor, it is clear that only the bubble diffuser contactor was able to achieve utilization of 4.8 mg/l. In order for the packed column and positive pressure injector to achieve comparable levels of ozone utilization, the concentration of ozone in the inlet gas (Y_1) must be raised.

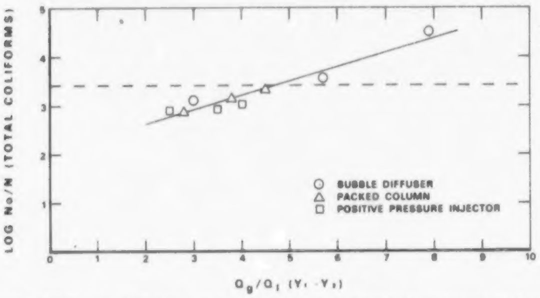

FIGURE 10. MEAN LOG TOTAL COLIFORM REDUCTION AS A FUNCTION OF OZONE UTILIZATION

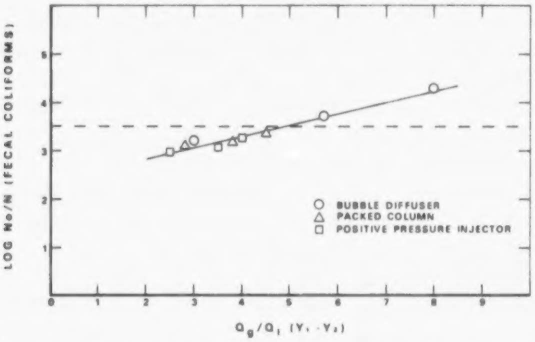

FIGURE 11. MEAN LOG FECAL COLIFORM REDUCTION AS A FUNCTION OF OZONE UTILIZATION

Effect of Increasing Y_1 at a Constant Q_G/Q_L Ratio

The objective of this experiment was to determine the change in percent ozone utilization, actual ozone utilization, and microorganism reduction in the three contactors when the applied dose was varied by changing the ozone concentration (Y_1) and maintaining a constant Q_G/Q_L ratio. The Q_G/Q_L ratio chosen for this experiment was 0.5 (37.5 l/min gas flow rate and 75 l/min liquid flow rate per contactor). The maximum Y_1 attainable with our ozone generator at this gas flow rate was 15 mg/l_{gas} (approximately 1.25 weight percent). Thus, the three concentration levels selected for this portion of the study were 3.0, 9.0, and 15.0 mg/l_{gas}, corresponding to applied dose levels of 1.5, 4.5 and 7.5 mg/l, respectively. The period during which this experiment was conducted was July 25 to August 16, 1978.

Quality of Wastewater

In Table 10, the minimum, mean, and maximum values of the filtered pilot plant effluent characteristics prior to ozonation are summarized for the second study period. There were several rainy days during this time period, which probably accounts for the lower readings relative to those of the first study period. Again, the filtered secondary effluent was of excellent quality.

Percent Ozone Utilization

In Table 11, the mean percent ozone utilization data per contactor at each dose level are summarized. The ANOVA for those data are presented in Table 12. In the whole plot analysis, the Block effect (B_i) was not significant, indicating very little day to day variation in percent utilization occurred in the contactors. However, C_j was highly significant ($p < 0.01$), indicating that overall percent ozone utilization was significantly different in at least two of the contactors (note column means in Table 11). To determine where the significant differences occurred,

TABLE 10. SUMMARY OF FILTERED PILOT PLANT EFFLUENT CHARACTERISTICS PRIOR TO OZONATION (JULY 25 to AUGUST 16, 1978)

Parameter	Mean	Minimum	Maximum
TCOD, mg/l	26	18	44
SCOD, mg/l	25	17	42
TOC, mg/l	8.1	4.4	13.0
TSS, mg/l	2.9	1.2	5.4
NH_4^+-N, mg/l	11.4	5.8	17.0
TKN, mg/l	12.5	6.2	18.9
Turbidity, mg/l	2.0	0.8	4.3
pH	—	7.4	8.1
Temperature, °C	23	22	23
Total Coliforms per 100 ml*	9.1 x 10^5 (geometric mean)	8.5 x 10^4	4.8 x 10^6
Fecal Coliforms per 100 ml**	2.7 x 10^5 (geometric mean)	2.0 x 10^4	1.7 x 10^6

* Adjusted for 87% colony verification

** Adjusted for 95% colony verification

TABLE 11. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)

	Mean Percent Ozone Utilization (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean % U, All Contactors
1.5	84 (4)	79 (3)	89 (2)	84 (5)
4.5	68 (5)	68 (4)	80 (4)	72 (7)
7.5	62 (4)	62 (3)	77 (3)	67 (8)
Mean % U, All Doses	71 (10)	69 (8)	82 (6)	

Tukey's HSD multiple comparison test was used. Results indicated that overall percent ozone utilization was higher in the bubble diffuser than in either the packed column or the positive pressure injector, while percent utilization in the latter two contactors was similar.

In the split plot analysis the Dose (D_j) effect was highly significant ($p < 0.01$) indicating that percent ozone utilization in all contactors varied with applied dose (note row means in Table 11). The C_jD_k interaction was also highly significant [($p < 0.01$), Table 12]. Thus, differences in percent ozone utilization between contactors were not the same at each dose level. The data from Table 11 are depicted graphically in Figure 12.

TABLE 12. ANOVA FOR PERCENT OZONE UTILIZATION DATA (INCREASING Y_1, CONSTANT Q_G/Q_L)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	101.67	12.71	3.18
Contactors (C_j) (adj. for blocks)	2	532.00	266.00	66.50*
Whole Plot Error (B_iC_j)	7	28.00	4.00	
Split Plot Analysis				
Doses (D_k)	2	2786.78	1393.39	254.31*
C_jD_k	4	173.78	43.44	7.44*
B_iD_k	16	87.67	5.48	0.94
$B_iC_jD_k$	14	81.78	5.84	
TOTAL	53			

*significant at the $p<0.01$ level

Note that the trend is similar to that which was observed when dose was increased by changing the Q_G/Q_L at a constant Y_1 (Figure 4). The rate of decrease in percent ozone utilization at higher doses was substantially less in the bubble diffuser than in the other two contactors. When C_j was re-evaluated statistically at each individual dose level, it was found that percent utilization in the bubble diffuser was significantly higher than in the other two contactors at each dose level studied. Percent utilization in the positive pressure injector was higher than in the packed

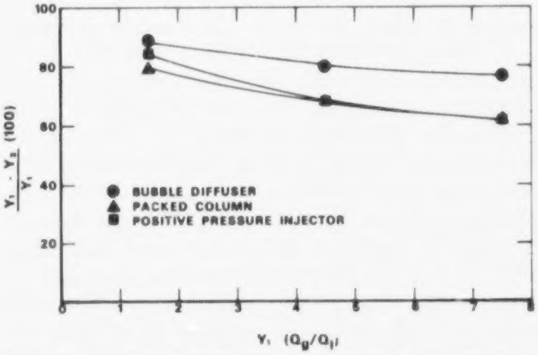

FIGURE 12. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)

column at the lowest dose level, but the two contactors were equivalent at the higher dose levels.

The B_iD_k interaction (Table 12) was not significant, indicating that the effect of dose levels on percent ozone utilization in all contactors was consistent from day to day. In Figure 13, the daily paired overall percent ozone utilization data are shown to illustrate the performance differences between contactors during the study period.

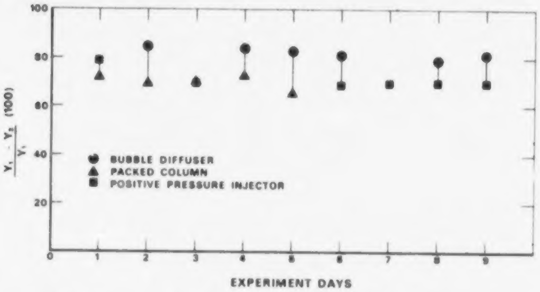

FIGURE 13. OVERALL MEAN PERCENT OZONE UTILIZATION PER REPLICATE RUN (EXPERIMENT 2)

Actual Ozone Utilization

In Table 13 the actual ozone utilization data (i.e., the amount of ozone transferred to the secondary effluent) are summarized. The ANOVA for those data is presented in Table 14. Results were similar to the percent ozone utilization data. In the whole plot analysis the B_i effect was not significant and the C_j effect was highly significant ($p < 0.01$). Tukey's HSD test indicated that overall ozone utilization was higher in the bubble diffuser than in the other two contactors. Overall ozone utilization in the packed column and positive pressure injector was similar.

In the split plot analysis, both D_k and C_jD_k were highly significant ($p < 0.01$), indicating that ozone utilization in at least two of the contactors was different, and that differences between contactors varied with applied ozone dose. The results are plotted in Figure 14, wherein it can be seen that the rate of increase in ozone utilization was higher in the bubble diffuser than in the other two contactors. Thus, the bubble diffuser, when operated in parallel with either the packed column or the positive pressure injector, had a greater capacity to absorb more ozone, and that capacity appears to have increased relative to the packed column and the positive pressure injector as more ozone was applied.

The B_iD_k interaction (Table 14) was significant ($p < 0.05$), indicating for the first time day-to-day differences in dose effects between contactors. The effluent quality was considerably more variable during this time period than in the previous period, due to moderately heavy rainfall on several experiment days. This most likely caused the significant B_iD_k effects.

Total Coliform Reduction

The total coliform log reduction (TCLR) data from this experiment are summarized in Table 15 and the ANOVA in Table 16. In the whole plot

TABLE 13. MEAN OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)

	Mean Ozone Utilization, mg/l (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean U, All Contactors
1.5	1.2 (0.08)	1.2 (0.04)	1.3 (0.1)	1.3 (0.09)
4.5	3.0 (0.2)	3.0 (0.2)	3.6 (0.2)	3.2 (0.3)
7.5	4.6 (0.3)	4.6 (0.3)	5.7 (0.2)	4.9 (0.6)
Mean U, All Doses	3.0 (1.4)	3.0 (1.4)	3.5 (1.8)	

TABLE 14. ANOVA FOR OZONE UTILIZATION DATA (INCREASING Y_1, CONSTANT Q_G/Q_L)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	0.26	0.038	3.34
Contactors (C_j) (adj. for blocks)	2	1.28	0.64	56.52**
Whole Plot Error ($B_i C_j$)	7	0.079	0.011	
Split Plot Analysis				
Doses (D_k)	2	123.03	61.52	1959.24**
$C_j D_k$	4	2.06	0.51	47.50**
$B_i D_k$	16	0.50	0.031	2.90*
$B_i C_j D_k$	14	0.15	0.011	
TOTAL	53			

* Significant at the $p < 0.005$ level

**Significant at the $p < 0.01$ level

analysis, neither the B_i effect nor the C_j' effect was significant. Thus, overall TCLR in all contactors was similar (note column means in Table 15). This effect, which at first may appear somewhat surprising, is not unexpected when one considers the magnitude of the overall differences in ozone utilization between contactors (column means, Table 13). Only 0.5 mg/l more ozone was absorbed in the bubble diffuser than in the other two contactors, and, although this was statistically significant, the increase in absorbed ozone was not sufficient to cause a marked increase in TCLR.

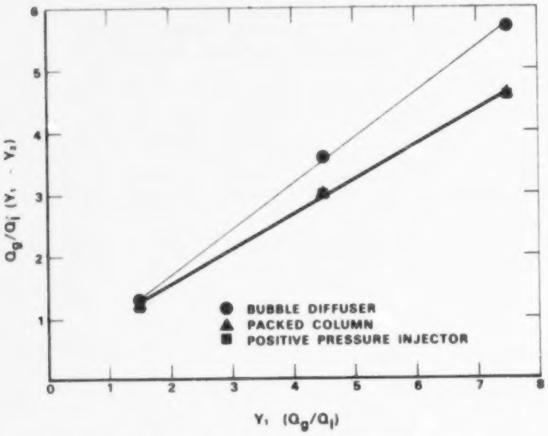

FIGURE 14. MEAN OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_g/Q_l)

In the split plot analysis (Table 16), the dose main effect (D_k), the contactor-dose interaction (C_jD_k), and the Block-Dose interaction (B_iD_k) were highly significant ($p < 0.01$). When the results were re-analyzed statistically at each dose level, it was found that all contactors elicited similar TCLR responses at the 1.5 and 4.5 mg/l applied dose levels. However, at the 7.5 mg/l level, TCLR in the bubble diffuser was significantly higher than in the packed column, while there was no difference between the packed column and the positive pressure injector. These effects are displayed graphically in Figure 15.

In this experiment, more day-to-day variation in the coliform reduction data occurred because of the variable effluent quality. This would most likely explain the significant B_iD_k interaction.

Fecal Coliform Reduction

Results from this experiment were almost exactly equivalent to the TCLR data. They are plotted in Figure 16. The fecal coliform log reduction data from this experiment are summarized in Table 17 and the ANOVA in Table 18.

TABLE 15. MEAN TOTAL COLIFORM LOG REDUCTION (TCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)

	Mean TCLR (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean TCLR, All Contactors
1.5	0.81 (0.68)	0.73 (0.59)	0.56 (0.40)	0.70 (0.54)
4.5	3.07 (0.43)	2.90 (0.39)	3.24 (0.49)	3.07 (0.44)
7.5	3.28 (0.40)	3.37 (0.30)	4.00 (0.49)	3.55 (0.50)
Mean TCLR, All Doses	2.39 (1.25)	2.34 (1.25)	2.60 (1.58)	

TABLE 16. ANOVA FOR TOTAL COLIFORM LOG REDUCTION DATA (INCREASING Y_1, CONSTANT Q_G/Q_L))

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot analysis				
Blocks (B_i) (adj. for contactors)	8	0.98	0.12	2.06
Contactors (C_j) (adj. for blocks)	2	0.062	0.031	0.52
Whole Plot Error (B_iC_j)	7	0.42	0.059	
Split Plot Analysis				
Doses (D_k)	2	83.94	41.97	132.82*
C_jD_k	4	1.69	0.42	6.14*
B_iD_k	16	5.06	0.32	4.59*
$B_iC_jD_k$	14	0.96	0.069	
TOTAL	53			

*significant at the $p < 0.01$ level.

Coliform Reduction vs. Ozone Utilization

Previously, we showed that, by plotting TCLR and FCLR vs. ozone utilization (Figures 10 and 11), equivalent coliform reductions were achieveable in a given wastewater, independent of contactor type and contact time. The increase in log reduction in Figures 10 and 11 was relatively flat, *following* the first 2 to 3 mg/l ozone utilized. This apparent "all-or-none" phenomenon suggested that perhaps log coliform reduction was a function of *log* ozone utilization. When all the

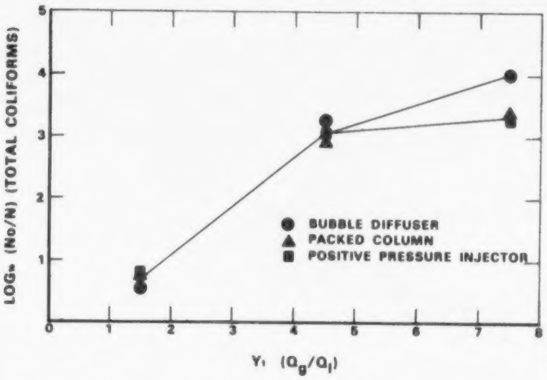

FIGURE 15. MEAN LOG TOTAL COLIFORM REDUCTION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_g/Q_l)

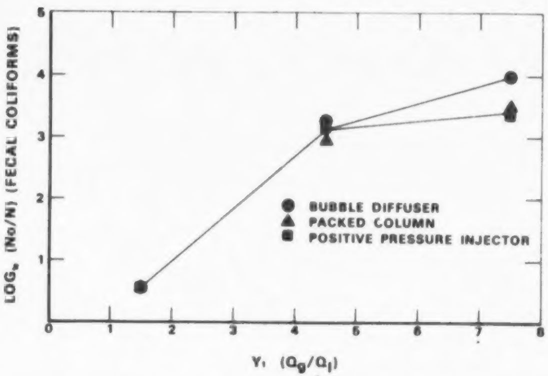

FIGURE 16. MEAN LOG FECAL COLIFORM REDUCTION REDUCTION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_g/Q_l)

TABLE 17. MEAN FECAL COLIFORM LOG REDUCTION (FCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)

	Mean FCLR (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean FCLR, All Contactors
1.5	0.58 (0.57)	0.58 (0.60)	0.54 (0.50)	0.57 (0.52)
4.5	3.17 (0.45)	2.94 (0.40)	3.27 (0.37)	3.13 (0.41)
7.5	3.37 (0.38)	3.48 (0.32)	4.00 (0.43)	3.62 (0.45)
Mean FCLR, All Doses	2.37 (1.38)	2.34 (1.37)	2.60 (1.59)	

TABLE 18. ANOVA FOR FECAL COLIFORM LOG REDUCTION DATA (INCREASING Y_1, CONSTANT Q_G/Q_L)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F
Whole Plot Analysis				
Blocks (B_i) (adj. for contactors)	8	0.60	0.075	1.68
Contactors (C_j) (adj. for blocks)	2	0.058	0.029	0.65
Whole Plot Error ($B_i C_j$)	7	0.31	0.045	
Split Plot Analysis				
Doses (D_k)	2	96.75	48.37	130.83**
$C_j D_k$	4	0.95	0.24	4.73**
$B_i D_k$	16	5.92	0.37	7.40**
$B_i C_j D_k$	14	0.70	0.050	
TOTAL	53			

**Significant at the $p<0.01$ level

ozone utilization data from *both* experiments were transformed to $\log_{10}$ values and plotted on the abscissa against the corresponding log coliform reduction data, the resulting graphs confirmed this hypothesis. The total coliform reduction data are depicted in Figure 17 and the fecal coliform data in Figure 18. A highly significant log-log correlation between coliform reduction and ozone utilization was found. The correlation coefficients for the total and fecal coliform data were 0.985 and 0.987, respectively (the intercepts were forced through the origin). Each figure includes 108 data points, gathered from both experiments, plus an additional 12 data points (indicated by triangular symbols) collected later for verification of the regression equations. Although it is possible that higher order relationships (cubic, quadratic, etc.) may exist in the data, we concluded that for all practical purposes the linear log-log relationship is sufficient as a predictive tool. For example, to achieve a final effluent total or fecal coliform density of 1000/100 ml or 200/100 ml, respectively, starting with an initial number of 1.55×10^6 total coliforms/100 ml or 4.17×10^5 fecal coliforms/100 ml (the respective geometric means of our data), 4.0 mg/l ozone must be absorbed by the effluent (dotted line labeled "a" in Figures 17 and 18). To achieve 70 total coliforms/100 ml or 14 fecal coliforms/100 ml in the final effluent, 6.5 mg/l

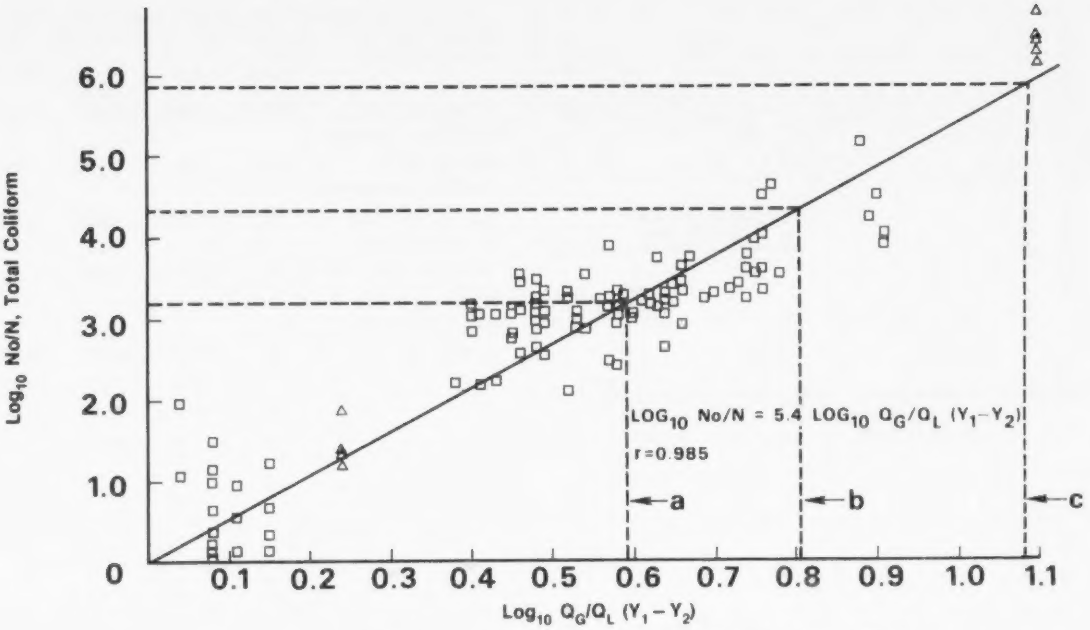

FIGURE 17. Mean Log Total Coliform Reduction as a function of Log Ozone Utilization

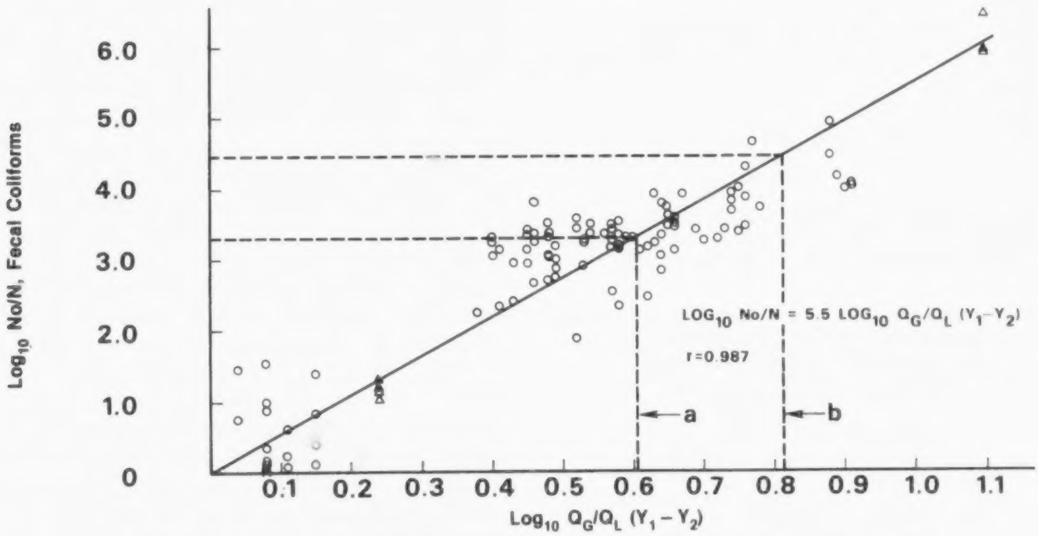

FIGURE 18. Mean Log Fecal Coliform Reduction as a function of Log Ozone Utilization

ozone must be absorbed (dotted line labeled "b" in Figures 17 and 18). Finally, to achieve the strict State of California standard of 2.2 total coliforms/100 ml, 12.1 mg/l ozone must be absorbed (dotted line labeled "c" in Figure 17). It is clear from the foregoing data presentation that only one of the three contactors studied would be capable of meeting all the above effluent limitations reliably, with ozone generated from air. That contactor is the bubble diffuser.

ACKNOWLEDGMENTS

The able efforts of Messrs. Glenn Gruber, Leo Fichter, and Burney Jackson in constructing the ozone disinfection pilot system are gratefully acknowledged.

Ms. Catherine Yeats and Messrs. Harold L. Sparks and Charles Palopoli assisted in the sampling, performance of ozone mass balances, and membrane filtration assays. Mr. Harold P. Clark identified, enumerated, and confirmed all coliform colonies. Chemical analyses were conducted by the Waste Identification and Analysis Section, Wastewater Research Division, Municipal Environmental Research Laboratory, U.S.E.P.A., Cincinnati, Ohio.

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DISCUSSION

DR. LONGLEY: I assume we can draw one more conclusion from what you said, talking about the bubble diffuser, and that is if you do not kill them right away you do not kill them at all. In other words, within thirty-five seconds.

MR. VENOSA: No. In the bubble diffuser, the residence time was 9.4 minutes.

DR. LONGLEY: I got it turned around. That was the positive pressure injector.

MR. VENOSA: Thirty-five seconds was the residence time in the packed column.

DR. LONGLEY: Okay, well then let's take the packed column at thirty-five seconds. All the kill then occurs in thirty-five seconds.

MR. VENOSA: Correct, but the kill was independent of detention time.

DR. LONGLEY: I would like to ask you another question and you may have covered this and I may have missed it. How did you do your ozone residual analysis?

MR. VENOSA: I did not mention anything about the residual, but we measured the residuals by the backward amperometric titration procedure.

DR. LONGLEY: Did you scrub the gas out before . . .

MR. VENOSA: We have done that. We have looked at the forward titration, the backward titration, and the ozone scrubbing method, and each one gives a different result. That is why I did not mention it.

DR. LONGLEY: If you are measuring directly in the liquid rather than scrubbing the gas, are you not really measuring total oxidants, at least the materials that oxidize the iodide? You are not really measuring ozone, are you?

MR. VENOSA: Presumably so, yes.

DR. ROSEN: You did say that for this particular work that these numbers you were comparing were at the end of the contact time, at which time you added thiosulphate or something to kill the reaction. You do not know that the residuals would not show a continued kill with time.

MR. VENOSA: We found that with the packed column we were capable of evaluating that effect. Remember those two 200 liter hold tanks that we had? When we enumerated coliforms at the end of those hold tanks versus the beginning of the hold tanks, there wasn't really much difference. It is when the water is actually in contact with the ozone in the contactors when you get your kill.

DR. ROSEN: Bollyky in the New York studies found that there was some holding time effect. Part of what I am trying to get at is to see if this regrowth or reactivation occurs with ozone as opposed to some of the other disinfectants.

MR. VENOSA: We have not looked at

regrowth. We have not looked at that with ozone.

QUESTION: I have a question on the residual. If you measured the residual, why didn't you include it in your mass balance when you were figuring ozone utilization in the columns?

MR. VENOSA: Residual is included in that. When you measure the off-gas concentration relative to the in-gas concentration, the difference is that which has been added to the wastewater, not necessarily consumed by the wastewater. Now if you look at the consumption of the ozone by the wastewater whatever is left is the residual.

QUESTION: Right, but then you are not getting a true kill per mg of ozone, if you do not subtract out the residual. You are saying that all the ozone is being utilized if you are only subtracting the off-gases.

MR. OPATKEN: That is the reason that the two hold tanks were placed after the packed column, which provides a true countercurrent operation. Normally if you are going to have a substantial residual, you would get it in a packed column. So, we put the two hold tanks in to provide additional residence time to consume the ozone and it did. So, there was six minutes of residence time there which consumed the ozone, and that also was then utilized. For the other contactors, the PPI operated cocurrently. You do not have the residual that you do in the countercurrent operation. In the diffuser the ozone is consumed as it is sitting in the tank itself for three minutes in each column. So you do not have a high residual coming off the bubble diffuser.

QUESTION: But even if you have 0.3 ppm at a low dosage of 3 ppm, that a 10% effect.

MR. OPATKEN: We did measure the residuals coming off the columns, and they are lower than that.

QUESTION: But if you used standard methods, you said you used the standard back titration method, and they say right in the standard method that they are not accurate down past ranges in that area.

MR. OPATKEN: True.

MR. VENOSA: What area?

QUESTION: Down past three parts per million, but if you use Schechter's spectrophotometric method that was published in 1973 it goes down past those ranges.

DR. HILL: I have a question about the method you use to measure the ozone concentration in your air stream. I think you mentioned that you used Birdsall's method. Is this one based on buffered solution of the iodide, or what method is this?

MR. VENOSA: The pH of the titration is performed at 2.

DR. HILL: When you are bubbling your ozone through the iodide solution, is the iodide solution neutral?

MR. VENOSA: Unbuffered.

MR. OPATKEN: There was a Dasibi monitor that corresponded exceedingly well with the wet test method. It was within plus or minus 2%.

DR. ROSEN: With respect to the Schechter method, that was done on very clean water as opposed to these kinds of waters. Then you would expect certainly the residuals to hang around long enough to get a reading in the spectrophotometer cell. Here we are talking about wastewater, and it does have some interferences and other things. So I am not sure how valuable, especially when you consider the low concentrations you are talking about, that Schechter's method is in wastewater.

DR. JOHNSON: I find it disturbing when we talk about ozone concentration in wastewater. As Longley pointed out at the beginning of the discussion, there is not any ozone in that wastewater. It is oxidant. One of the reasons you do not see the classical kind of time dependence of disinfection is that there is ozone there but just for a very short period of time. When you measure with the KI right in the sewage you are not measuring ozone even though Schechter says it is ozone.

MR. OPATKEN: We have stripped it out as it was coming into our sample, using a glass distillation flask. Now we cannot do it stoichiometrically, but we do know that we have ozone residual, because we strip it out and then we bubble it into potassium iodide. So we do have an ozone residual.

DR. JOHNSON: What are your comparisons? You are talking about some new data we have not seen. What is the comparison between the ozone that you measure in the solution and the ozone that you measure in the stripped out ozone?

MR. OPATKEN: The ozone stripped out may be 50 to 60% of that which we meausre by the back titration method. That 50 to 60% is not sacred either, but that is the reason we just have not been able to define ozone residuals as such, but we do have ozone residuals by the stripped method with wastewater.

DR. JOHNSON: After how inuch time?

MR. OPATKEN: No time. As we take our liquid sample we are stripping. In other words, we have gas flowing and stripping out the ozone when we start our liquid sampling.

DR. JOHNSON: You are pumping in ozone gas.

MR. OPATKEN: No. We are putting liquid in and pumping in dry air, and stripping that ozone that is in the wastewater from the wastewater.

19.

ECONOMIC EVALUATION OF OZONE CONTACTORS

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INTRODUCTION

This paper discusses the economics of disinfection in three different contactors. The evaluation is based heavily on pilot plant results (1). Detailed cost sheets and summary cost data have been developed on the three contactors. The cost data are summarized in the accompanying 18 tables.

Conditions and Assumptions

The ground rules that were employed during this evaluation are:

1. This paper compares the economics of disinfecting wastewater with ozone using the following contactors:
   a. Packed Column (PC)
   b. Positive Pressure Injector (PPI)
   c. Bubble Diffuser operating at:
      (1) 5 mg/l O_3 concentration.
      (2) 10 mg/l O_3 concentration.
      (3) 15 mg/l O_3 concentration.
2. The evaluation is based on pilot plant data and extrapolated to one treatment plant size. That size is 5000 m^3/d.
3. The data are in metric units. English units are displayed in the paper within parentheses. In the tabular material the roman numeral "M" refers to 1000 throughout the paper.
4. The process selected for disinfection may not be the most economical, but the costs are used to compare contactors, not optimize ozonation. An illustration might be helpful. To produce ozone from air, the air should be dried to below −40°C dew point. The process employed at the pilot plant consisted of compressing ambient air to 700 kPa and cooling the compressed air to ambient temperature. This pressurization and cooling lowered the dew point to −7°C which removed 87% of the moisture. The compressed air was then dried to below −40°C by adsorbing water vapor on an alumina bed. Approximately 25% of the dry air stream was passed through a second bed of alumina to regenerate depleted alumina beads and thus maintain a continuous flow system. This probably was not the most efficient method for drying air. However, since it was the method used at the pilot plant, it will serve as the basis for the ensuing cost analysis. The individual power cost of compressing and regenerating the air can be compared with other methods of drying, but the prime purpose of this paper is to compare costs associated with contacting, not drying. Since the air was dried by the same technique for all contactors, the comparative evaluation is valid and differential costs would only be slightly affected. The same is true for the ozonator. Ozone generation power costs were based on the pilot plant ozonator. If there are ozonators that require less power at these concentrations, and capital costs are about the same as the pilot plant unit, then there could be a reduction in the cost of disinfection, but it probably will not significantly affect the differential costs between the contactors.
5. In reporting operating and overhead costs, the assumptions that were used in calculating these unit costs are given. These values are estimates and as such they are not sacred. However, the values on power costs or material costs, such as antifoam, are based on actual costs, and for this particular wastewater they are firm.
6. The unit cost for producing ozone was

TABLE 1. SUMMARY OF CONTACTOR EVALUATION

	DIFFUSER			PACKED COLUMN	PRESSURE INJECTOR
Concentration (mg/l) O_3	5	10	15	15	21
Capital Investment ($)	406,000	266,000	336,000	445,000	665,000
Disinfection Cost ($/Mm^3)	39.4	29.2	34.4	47.3	60.00
Disinfection Cost (¢/Mgal)	15	11	13	18	23

developed and then carried over to the disinfection cost.

Results and Discussion

Table 1 is a summary of the contactor evaluation. The diffusers were studied at three different ozone gas concentrations: 5, 10, and 15 mg/l. The packed column and pressure injector were evaluated in parallel with the diffuser.

The diffuser at 10 mg/l had the lowest capital investment, i.e., $266,000. Capital costs range from this minimum to $665,000. The diffuser at 10 mg/l also had the lowest disinfection cost, which was $29.20/Mm^3 or 11¢/Mgal. These values include operating and overhead costs. Since this evaluation is scaled to a treatment plant size of 5000m^3/d, it can be expected that operating labor and amortization costs contribute a disproportionate share of the total disinfection cost.

OZONE COST SHEETS

Listed on Table 2 are the bases for determining the quantity and concentration of ozone. These data were developed at the pilot plant.

With the information from Table 2 the individual cost sheets can be developed. The ozone costs for the packed column are given on Table 3. The direct out-of-pocket cost for ozone is 99¢/kg or 45¢/lb. The rest of the cost items are dependent either on production or the fixed capital investment (FCI). The significant items are the operating labor and especially the amortization cost for overhead. Putting these values into the estimate yields a total treatment cost for generating ozone of $4.00/kg or $1.82/lb. The FCI was determined by the number of units required to produce this quantity of ozone at the concentration required. In the case of the packed column the number of units was 39. A unit consists of six cells. The unit was mapped to determine concentration vs. power @ various gas flows (Figure 1), then scaled up accordingly.

On Table 4 is the unit cost for ozone produced for the pressure injector. In this case, the cost is $5.23/kg or $2.38/lb. It should be pointed out that there are significant differences between the estimates on the PC and PPI. A significant difference is the ozone generation power. To produce ozone at a concentration of 21 mg/l requires considerably more power that it does to produce a concentration of 15 mg/l and this is the reason for the increased power cost. This cost sheet shows that the unit operating labor is less for the PPI. This is caused by the higher ozone requirement with the PPI. The PPI requires 50 kg O_3/d whereas the PC requires 42 kg/d. However, any apparent operating cost advantage will be lost when disinfection is taken into account. The higher overhead costs for the PPI are

TABLE 2. BASIS OF OZONE REQUIREMENTS

- Ozone utilized to achieve disinfection is 4 mg/l for all contactors
- Safety factor of 20% on ozone utilization or 5 mg/l absorbed
- PPI required an ozone concentration of 21 mg/l at an applied dose of 10 mg/l to achieve an absorbed dose of 5 mg/l. % U = 50%
- Packed column required an ozone concentration of 15 mg/l at an applied dose of 8.3 mg/l to achieve an absorbed dose of 5 mg/l. % U = 60%
- Diffuser @ 5 mg/l concentration required an applied dose of 6.2 mg/l to achieve an absorbed dose of 5 mg/l. % U = 80%
- Diffuser @ 10 mg/l concentration required an applied dose of 5.9 mg/l to achieve an absorbed dose of 5 mg/l. % U = 85%
- Diffuser @ 15 mg/l concentration required an applied dose of 5.6 mg/l to achieve an absorbed dose of 5 mg/l. % U = 90%
- Ozone mapping curve was used to determine capacity and power for generation
- Cost sheets were developed on the cost of ozone
- Cost sheets on disinfection used the unit cost of ozone

TABLE 3. OZONE COST SHEET - PACKED COLUMN

O_3 Rate = 42 kg/d @ 15 mg/l					
FCI = $370,000					
I. Operating Cost					
A. Utility Cost					
1. Electric Power	Ratio kWh/kg	Price ¢/kWh	Cost ¢/kg		
a. Compression	5	3	15		
b. Regeneration	1	3	3		
c. Generation	26	3	78		
2. Water	Mgal/kg	¢/Mgal			
a. Cooling	.3	10	3		
Total Utility Cost (¢/lb)				99 (45)	
B. Operating Labor (1/2 MY)			41		
C. Repair Labor (1% FCI)			24		
D. Supervision [15% (O.L. + R.L.)]			10		
E. Repair Materials (1% FCI)			24		
F. Supplies [10% (O.L. + R.L. + Supv)]			8		
Total Operating Cost				206	
II. Overhead Cost					
A. Ins. (1% FCI)			24		
B. Amortization (7% FCI)			170		
Total Overhead Cost				194	
TOTAL OZONE COST (¢/lb)					400 (182)

attributed to the high fixed capital investment. In the case of the PPI, 83 ozone units are required to obtain a production of 50 kg/d @ 21 mg/l, whereas 39 units were required for the PC, since only 42 kg/d @ 15 mg/l O_3 concentration were required.

Tables 5, 6, and 7 cover the ozone costs for the bubble diffuser, at the three ozone concentrations. The quantity of ozone required for the bubble diffuser is considerably less than it is for the PC or PPI. This is the result of a significantly higher utilization efficiency. The utilization is 80, 85 and 90% for the diffuser at ozone concentrations of 5, 10, and 15 mg/l, respectively, and only 60% for the PC and 50% for the PPI.

The three different bubble diffuser cases show the effect of concentration on ozone generation power: as the concentration increases so does the power to generate ozone. For 5 mg/l the generation power is 15 kwh/kg, at 10 mg/l the generation power increases to 21 kwh/kg, and at 15 mg/l the power is 26 kwh/kg. However, as the ozone concentration decreases, the volume of air required for compression and drying increases to obtain the necessary dosage. The advantage of lower power generation is lost in air preparation, in that 35 kwh/kg is the total power required at 5 mg/l and 32 kwh/kg is the total power at 15 mg/l. The major differences in cost for the three operating conditions on the diffuser are related to the overhead differences caused by the FCI for the three cases. The numbers of ozone units required to achieve the production level of 31 kg/d are 37 for 5 mg/l; 18 to produce 29 kg/d at 10 mg/l; and 26 units to produce 28 kg/d at 15 mg/l.

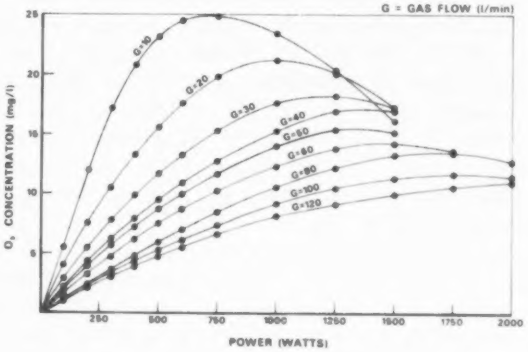

Figure 1. Ozone Concentration vs. Power at Specified Gas Flow Rates

TABLE 4. OZONE COST SHEET - PPI

O_3 Rate = 50 kg/d @ 21 mg/l
FCI = $630,000

	Ratio	Price	Cost		
I. Operating Cost					
A. Utility Cost					
1. Electric Power	kWh/kg	¢/kWh	¢/kg		
a. Compression	4	3	12		
b. Regeneration	1	3	3		
c. Generation	35	3	105		
2. Water	Mgal/kg	¢/Mgal			
a. Cooling	.2	10	2		
Total Utility Cost (¢/lb)				122 (55)	
B. Operating Labor (1/2 MY)			34		
C. Repair Labor (1% FCI)			36		
D. Supervision [15% (O.L. + R.L.)]			11		
Repair Materials (1% FCI)			36		
F. Supplies [10% (O.L. + R.L. + Supv)]			8		
Total Operating Cost				247	
II. Overhead Cost					
A. Ins. (1% FCI)			36		
B. Amortization (7% FCI)			240		
Total Overhead Cost				276	
TOTAL OZONE COST (¢/lb)					523 (238)

TABLE 5. OZONE COST SHEET - DIFFUSER

O_3 Rate = 31 kg/d @ 5 mg/l
FCI = $350,000

	Ratio	Price	Cost		
I. Operating Cost					
A. Utility Cost					
1. Electric Power	kWh/kg	¢/kWh	¢/kg		
a. Compression	16	3	48		
b. Regeneration	4	3	12		
c. Generation	15	3	45		
2. Water	Mgal/kg	¢/Mgal			
a. Cooling	.8	10	8		
Total Utility Cost (¢/lb)				113 (51)	
B. Operating Labor (1/2 MY)			55		
C. Repair Labor (1% FCI)			32		
D. Supervision [15% (O.L. + R.L.)]			13		
E. Repair Materials (1% FCI)			32		
F. Supplies [10% (O.L. + R.L. + Supv)]			10		
Total Operating Cost				255	
II. Overhead Cost					
A. Ins. (1% FCI)			32		
B. Amortization (7% FCI)			220		
Total Overhead Cost				252	
TOTAL OZONE COST (¢/lb)					507 (230)

TABLE 6. OZONE COST SHEET - DIFFUSER

O_3 Rate = 29 kg/d @ 10 mg/l

FCI = $210,000

	Ratio	Price	Cost		
I. Operating Cost					
A. Utility Cost					
1. Electric Power	kWh/kg	¢/kWh	¢/kg		
a. Compression	8	3	24		
b. Regeneration	2	3	6		
c. Generation	21	3	63		
2. Water	Mgal/kg	¢/Mgal			
a. Cooling	.4	10	4		
Total Utility Cost (¢/lb)				97 (44)	
B. Operating Labor (1/2 MY)			59		
C. Repair Labor (1% FCI)			18		
D. Supervision [15% (O.L. + R.L.)]			12		
E. Repair Materials (1% FCI)			18		
F. Supplies [10% (O.L. + R.L. + Supv)]			9		
Total Operating Cost				213	
II. Overhead Cost					
A. Ins. (1% FCI)			18		
B. Amortization (7% FCI)			130		
Total Overhead Cost (¢/lb)				148	
TOTAL OZONE COST					361 (164)

TABLE 7. OZONE COST SHEET - DIFFUSER

O_3 Rate = 28 kg/d @ 15 mg/l

FCI = $280,000

	Ratio	Price	Cost		
I. Operating Cost					
A. Utility Cost					
1. Electric Power	kWh/kg	¢/kWh	¢/kg		
a. Compression	5	3	15		
b. Regeneration	1	3	3		
c. Generation	26	3	78		
2. Water	Mgal/kg	¢/Mgal	¢/kg		
a. Cooling	.3	10	3		
Total Utility Cost (¢/lb)				99 (45)	
B. Operating Labor (1/2 MY)			61		
C. Repair Labor (1% FCI)			30		
D. Supervision [15% (O.L. + R.L.)]			14		
E. Repair Materials (1% FCI)			30		
F. Supplies [10% (O.L. + R.L. + Supv)]			10		
Total Operating Cost				244	
II. Overhead Cost					
A. Ins. (1% FCI)			30		
B. Amortization (7% FCI)			200		
Total Overhead Cost				230	
TOTAL OZONE COST (¢/lb)					474 (215)

TABLE 8. SUMMARY OF OZONE COST

Contactor	Diffuser			Packed Column	Pressure Injector
Concentration (mg/l)	5	10	15	15	21
Fixed Capital Inv. ($)	350,000	210,000	280,000	370,000	630,000
Ozone Gen. Units	37	18	26	39	83
Ozone Production kg/d	31	29	28	42	50
O_3 Utility Cost ($/kg) ($/lb)	1.13 0.51	0.97 0.44	0.99 0.45	0.99 0.45	1.22 0.55
O_3 Total Cost ($/kg) ($/lb)	5.07 2.30	3.61 1.64	4.74 2.15	4.00 1.82	5.23 2.38
Gas Flow Rate (m^3/d)	6,200	2,950	1,870	2,780	2,380

Table 8 summarizes some of the significant differences:

1. The diffuser at an ozone concentration of 10 mg/l has a decided edge over the other conditions and contactors evaluated. The FCI and the ozone disinfection cost are at a minimum for both conditions.
2. The second best contactor appears to be the PC because of an advantage from an ozone cost standpoint. However, the advantage may be tied in with low unit operating costs due to the higher production levels of ozone. When disinfection comes into play, the advantage for the PC will be offset by a higher dose and therefore higher disinfection costs.
3. The ozone cost between $1.64 and $2.38 per pound is expensive because of the low production rate. To keep this chemical, ozone, in perspective, remember that 30,000 lb/yr is considerably different from chlorine plants that have capacities in the tons/day category. And this is really why the cost difference is significant.

DISINFECTION COST SHEETS

Packed Column

With the ozone costs established, these values were then applied to the various contactors to determine the cost of disinfection. Table 9 shows the costs for disinfecting with the packed column. The English equivalents in ¢/Mgal are within parentheses and individual cost items that are out-of-pocket costs, chiefly power costs, are given. Treating the costs in this fashion indicates that 80% of the cost of disinfection is associated with materials, and in this case, ozone. The operating costs account for 13% and overhead for 7%. The FCI for the packed column is only 17% of the FCI for the combined facility of ozonation plus disinfection. Notice that antifoam is included under material costs. It is a requisite if the packed column is to operate adequately; otherwise, high pressure drops and low effluent flow rates will occur.

Positive Pressure Injector

Comparing the packed column with the PPI, a significant difference lies in the material cost (Table 10). The overhead cost again plays a minor part in this evaluation since the FCI of the PPI contactor is only $35,000.

Bubble Diffuser

In comparing the bubble diffuser at 3 ozone concentrations (Tables 11, 12, and 13), the concentration of 10 mg/l has the lowest disinfection cost. In these three cases, notice that the material cost is the only item affected since the operating and overhead costs are based on identical Mm^3 and the FCI is the same for all three cases.

It was stated earlier that the ozone cost for the packed column was second to the diffuser at 10 mg/l concentration. Notice that the advantage of increased ozone production with the PC which

TABLE 9. DISINFECTION COST SHEET - PC AT 15 mg/l

(1) Treatment Plant Size = 5000 m^3/d
(2) Dose at 8.4 mg/l

I. Materials	Ratio kg/Mm^3	Price \$/kg	Cost \$/Mm^3		
A. Ozone	8.4	4.00	33.6		
B. Antifoam (¢/Mgal)	2.0	2.05	4.10(1.6)		
Total Materials cost				37.7	
II. Operating Cost	kW/Mm^3	¢/kW			
A. Power	36.5	3			
1. Electric			1.10		
B. Operating Labor (1/2 MY)			3.42		
C. Maintenance (1% FCI)			.41		
D. Supv. [15% (O.L. + M.L.)]			.57		
E. Maintenance Materials (1% FCI)			.41		
F. Supplies [10% (O.L. + M.L. + Supv)]			.44		
Total Operating Cost				6.35	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.41		
B. Amortization (7% FCI)			2.88		
Total Overhead Cost				3.29	
TOTAL DISINFECTION COST (¢/Mgal)					47.34 (18)

TABLE 10. DISINFECTION COST SHEET - PPI AT 21 mg/l

(1) Treatment Plant Size = 5000 m^3/d

(2) Dose at 10 mg/l

I. Materials	Ratio kg/Mm^3	Price \$/kg	Cost \$/Mm^3		
A. Ozone	10	5.23	52.3		
II. Operating Cost					
A. Power	kW/Mm^3	¢/kW			
1. Electric	.486	3	1.46		
B. Operating Labor (1/2 MY)			3.42		
C. Maintenance (1% FCI)			.19		
D. Supv. [15% (O.L. + M.L.)]			.54		
E. Maintenance Materials (1% FCI)			.19		
F. Supplies [10% (O.L. + M.L. + Supv)]			.42		
Total Operating Cost				6.22	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.19		
B. Amortization (7% FCI)			1.34		
Total Overhead Cost				1.53	
TOTAL DISINFECTION COST (¢/Mgal)					60.05 (23)

TABLE 11. DISINFECTION COST SHEET -DIFFUSER AT 5 mg/l

(1) Treatment Plant Size = 5000 m^3/d
(2) Dose at 6.2 mg/l

I. Materials	Ratio kg/M^3	Price \$/kg	Cost \$/Mm^3		
A. Ozone	6.2	5.07	31.43		
II. Operating Cost					
A. Power	kW/Mm^3	¢/kW			
1. Electric	.146	3	.44		
B. Operating Labor (1/2 MY)			3.42		
C. Maintenance (1% FCI)			.31		
D. Supv. [15% (O.L. + M.L.)]			.56		
E. Maintenance Materials (1% FCI)			.31		
F. Supplies [10% (O.L. + M.L. + Supv.)]			.43		
Total Operating Cost				5.47	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.31		
B. Amortization (7% FCI)			2.15		
Total Overhead Cost				2.46	
TOTAL DISINFECTION COST (¢/Mgal)					39.36 (15)

yielded reduced unit cost of ozone is significantly increased by the increased dose of 8.4 mg/l. The material costs jumped because of the higher ratio of kg/Mm^3 which negated the effect of a lower unit cost of ozone. The disinfection costs showed only a slight effect by the increased FCI of the PC and no difference in the operating labor cost since the base unit is identical in terms of Mm^3.

SUMMARY OF DISINFECTION COST

Table 14 shows the applied dose as the major cause for the cost differences. The doses shown were established at the pilot plant. There are differences in contactor FCI, but these are relatively small, especially when compared with the FCI for ozonation. Finally, the diffuser at 10 mg/l is the most economical process for achieving disinfection with ozone.

TABLE 12. DISINFECTION COST SHEET · DIFFUSER AT 10 mg/l

(1) Treatment Plant Size = 5000 m^3/d
(2) Dose at 5.9 mg/l

I. Materials	Ratio kg/Mm^3	Price \$/kg	Cost \$/Mm^3		
A. Ozone	5.9	3.61	21.3		
II. Operating Cost					
A. Power	kW/Mm^3	¢/kW			
1. Electric	.146	3	.44		
B. Operating Labor (1/2 MY)			3.42		
C. Maintenance (1% FCI)			.31		
D. Supv. [15% (O.L. + M.L.)]			.56		
E. Maintenance Materials (1% FCI)			.31		
F. Supplies [10% (O.L. + M.L. + Supv.)]			.43		
Total Operating Cost				5.47	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.31		
B. Amortization (7% FCI)			2.15		
Total Overhead Cost				2.46	
TOTAL DISINFECTION COST (¢/Mgal)					29.23 (11)

TABLE 13. DISINFECTION COST SHEET - DIFFUSER AT 15 mg/l

(1) Treatment Plant Size = 5000 m^3/d
(2) Dose at 5.6 mg/l

	Ratio	Price	Cost		
I. Materials	kg/Mm^3	\$/kg	\$/Mm^3		
A. Ozone	5.6	4.74	26.5		
II. Operating Cost					
A. Power	kW/Mm^3	¢/kW			
1. Electric	.146	3	.44		
B. Operating Labor (1/2 MY)			3.42		
C. Maintenance (1% FCI)			.31		
D. Supv. [15% (O.L. + M.L.)]			.56		
E. Maintenance Materials (1% FCI)			.31		
F. Supplies [10% (O.L. + M.L. + Supv.)]			.43		
Total Operating Cost				5.47	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.31		
B. Amortization (7% FCI)			2.15		
Total Overhead Cost				2.46	
TOTAL DISINFECTION COST (¢/Mgal)					34.43 (13)

TABLE 14. SUMMARY OF DISINFECTION COSTS

	Diffuser			Packed Column	Pressure Injector
Concentration (mg/l)	5	10	15	15	21
Applied Dose (mg/l)	6.2	5.9	5.6	8.4	10
Utilized Dose (mg/l)	5	5	5	5	5
Capital Investment (\$)	56,000	56,000	56,00	75,000	35,000
Treatment Plant Size (m^3/d)	5,000	5,000	5,000	5,000	5,000
(mgd)	1.3	1.3	1.3	1.3	1.3
Disinfection Cost (\$/Mm^3)	39.4	29.2	34.4	47.3	60.00
(¢/Mgal)	15	11	13	18	23
Disinfection Cost-Direct (\$/Mm^3)	7.45	6.16	5.98	13.5	13.7
(¢/Mgal)	2.8	2.3	2.3	5.1	5.2

The last line of the summary indicates the out-of-the-pocket costs. These values range between 2.3 and 5.2¢/Mgal, which is more typical of published ozone costs. However, the problem arises that with this type of evaluation we could select the diffuser at 15 mg/l as an economical processing condition.

To place these costs in proper perspective a yearly budget should be considered. Tables 15 and 16 show the proposed yearly budget for the diffuser at 10 mg/l as an ozone budget and a disinfection budget, respectively. These two are combined on Table 17 to show the overall budgeted items. Amortization accounts for 33% of the budget and operating labor at 25%. The out-of-the-pocket cost or in this case, electrical power, constituted approximately 20% of the total budget. It is important to single out these individual cost items to determine where future research efforts should concentrate in order to reduce disinfection costs. In this case a program on reducing the FCI would prove more beneficial than reducing the power cost associated with generating ozone.

This type evaluation can be used to determine the cost of chlorine disinfection. Such an evaluation appears in Table 18. The cost is 5.1¢/Mgal. The out-of-pocket cost is 1.6¢/Mgal as against 2.3¢/Mgal for ozone. Assumptions included a dose of 10 mg/l and a contact time of 30 minutes.

TABLE 15. OZONE BUDGET FOR DIFFUSER @ 10 mg/l

O_3 Required = 29 kg/d ≈ 10,600 kg/yr

	$/yr		
I. Operating Budget			
A. Electric Power	9,800		
B. Cooling Water	400		
C. Operating Labor	6,200		
D. Repair Labor	1,900		
E. Supervision	1,200		
F. Repair Materials	1,900		
G. Supplies	900		
Total Operating Budget		22,300	
II. Overhead Budget			
A. Insurance	1,900		
B. Amortization	13,800		
Total Overhead Budget		15,700	
TOTAL OZONE BUDGET			38,000

TABLE 16. DISINFECTION BUDGET FOR DIFFUSER @ 10 mg/l

WW Rate = 5 Mm^3/d ≈ 1,800 Mm^3/yr

	$/yr		
I. Material Budget			
A. Ozone		38,000	
II. Operating Budget			
A. Electrical Power	800		
B. Operating Labor	6,200		
C. Repair Labor	600		
D. Supervision	1,000		
E. Repair Materials	600		
F. Supplies	800		
Total Operating Budget		10,000	
III Overhead Budget			
A. Insurance	600		
B. Amortization	3,900		
Total Overhead Budget		4,500	
TOTAL DISINFECTION BUDGET			52,500

TABLE 17. COMBINED OZONE AND DISINFECTION BUDGET FOR DIFFUSER @ 10 mg/l

WW = 1,800 Mm^3/yr

O_3 = 10,600 kg/yr

	$/yr		
I. Operating Budget			
A. Electric Power	10,600		
B. Cooling Water	400		
C. Operating Labor	12,400		
D. Repair Labor	2,500		
E. Supervision	2,200		
F. Repair Materials	2,500		
G. Supplies	1,700		
Total Operating Cost		32,300	
II. Overhead Budget			
A. Insurance	2,500		
B. Amortization	17,700		
Total Overhead Cost		20,200	
TOTAL OZONE DISINFECTION COST			52,500

TABLE 18. DISINFECTION COST SHEET - CHLORINE

(1) Treatment Plant Size = 5000 m^3/d
(2) Dose at 10 mg/l
(3) FCI = $80,000

	Ratio kg/Mm^3	Price $/kg		Cost $/$Mm^3$	
I. Materials					
A. Chlorine (¢/Mgal)	10	.41		4.10	(1.6)
II. Operating Cost					
A. Power			.44		
B. Operating Labor (1/2 MY)			3.42		
C. Repair Labor (1% FCI)			.44		
D. Repair Materials (1% FCI)			.44		
E. Supv. [15% (O.L. + R.L.)]			.58		
F. Supplies [10% (O.L. + R.L. + Supv.)]			.44		
Total Operating Costs				5.76	
III. Overhead Cost					
A. Ins. & Taxes (1% FCI)			.44		
B. Amortization (7% FCI)			3.07		
Total Overhead Costs				3.51	
TOTAL DISINFECTION COST (¢/Mgal)				13.37	(5.1)

REFERENCE

1. Venosa, A.D., M.C. Meckes, E.J. Opatken, and J.W. Evans, "Comparative Efficiencies of Ozone Utilization and Microorganism Reduction in Different Ozone Contactors." This Symposium.

DISCUSSION

MR. GIAIMO: ***Pure Water Systems:*** It was pointed out this morning that we reached a certain disinfection level at a certain cost. What disinfection level was achieved at these costs?

MR. OPATKEN: 200 fecal coliforms/100 ml. We said 200 was achieved at 4 mg/l, and our safety factor was applied on to that to make it 5 mg/l.

MR. GIAIMO: Did you have a corresponding increase to achieve the 2.2 MPN level?

MR. OPATKEN: No.

MR. GIAIMO: In other words you are saying that it cost the same to get 2.2 as it cost to get . . .

MR. OPATKEN: No. You want the cost for 2.2 MPN. I do not know. Al said he has one point that he got for you this morning. He said it took 15 mg/l, but I think he wants to check that point a lot more thoroughly than that.

MR. GIAIMO: That would be around 3 times the cost, then? In other words, if you have 5 mg/l to get 200 fecal coliforms, and you have to go to 15 mg/l to get 2.2, you're about 3 times the cost . . .

MR. OPATKEN: No. I haven't calculated that.

MR. GIAIMO: You are saying really you did not look at it.

MR. OPATKEN: At 2.2, no. I did not look at 2.2.

DR. RICE: Question and a comment. I did not get the amortization time on the ozonation system. It looks like it is about three years.

MR. OPATKEN: No, I checked with Bob Smith, and we rounded it off at 7% for 20 years. I think it was 6⅝ or 6⅞.

DR. RICE: I did not know whether the 7% meant interest on the money and I wanted to get the years. The second thing is, the half a man year. In the European drinking water plants they are using ozone or $Cl0_2$. These plants effectively run themselves. You do not have a half a man operating those things.

MR. OPATKEN: I think Dr. Jain will actually increase this estimate when he discusses start-up at the Meander facility, and he may use up his ten year contingent of manpower just for start-up.

DR. RICE: The third thing I would like to say is you have based all of your ozone generation production rate versus power on minus 40°C air, and mostly in European plants it is generally around minus 60°C. Minus 40°C in those generators over there seems to be just skirting the edge of getting too little ozone for the power.

MR. MECKES: We did dry our air below minus 40°C. However, it did vary. We did have some days when we were below minus 60°C as far as drying.

DR. ROSEN: I am obviously not going to be able to sit here and take all the numbers that you took and calculate the same sorts of things that you did, but I have done enough work in this area in making cost estimates to know that you are off by at least a factor of two. First of all, the particular unit that you are using to generate ozone is a one-of-a-kind non-production unit.

MR. OPATKEN: And have you got a curve of your unit to compare with that one?

DR. ROSEN: Yes. There are certain proprietary reasons why those things are not published, not because we do not know them. But you are talking about a one of a kind unit. Your generator is very much limited on the cooling capacity and some other things. Aside from that you referenced to the paper that Scaccia and I presented on contactors, and you basically came up with the same conclusion, that independent of the contactor you can get the same kind of disinfection if you get the same kind of utilized dose.

MR. OPATKEN: I think that is where we do agree.

DR. ROSEN: Now where we disagree, which is very important, is that in that same paper we showed that for a positive pressure injector operating somewhere near 6 mg/l there was something like 80% utilization, and for a diffuser system like you were using something like 98% utilization.

MR. OPATKEN: What was your gas-to-liquid ratio?

DR. ROSEN: It was a range, and that was all reported in that paper.

MR. OPATKEN: What was the concentration?

DR. ROSEN: It was around 1% by weight, 1 to 2% depending whether it was generated from air or oxygen.

MR. OPATKEN: We received the positive pressure injector from you.

DR. ROSEN: Yes, but things have changed. What you are saying is that prototypes are what ultimately become commercial products, and that is wrong. You are talking about a field that is changing very rapidly from day to day.

MR. OPATKEN: They are not changing that fast, Harvey.

20.

FIELD SCALE EVALUATION OF WASTEWATER DISINFECTION BY OZONE GENERATED FROM AIR

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ABSTRACT

The upper Thompson Sanitation district (UTSD) AWT facility located at Estes Park, Colorado, incorporated one of the first full-scale ozone wastewater disinfection systems in the United States. Plant design flow is 5,680 cu m/day (1.5 mgd) and current operating flows have ranged from 1,140 cu m/day to 3,790 cu m/day (0.3 mgd to 1.0 mgd). The ozone system has been operational for 2 years and good disinfection performance has occurred, but several required modifications have resulted in intermittent operation.

The ozone generation system is air-fed. The air pretreatment unit performed well, but periodic malfunctions have damaged the ozone generator. A high dew point alarm is being investigated to prevent this problem from reoccurring. The dew point of air from the pretreatment unit increased during an 8-hour drying tower cycle by approximately 20° C, and during normal operation all readings were better than the manufacturer's rated value. The increase in dew point resulted in as much as a 30 percent decrease in ozone production, which resulted in a decrease in the ozone dosage to the wastewater. Air pretreatment system design should address the fluctuation in air dew point.

Ozone power utilization (kwh/lb) was significantly greater at lower ozone production levels due to the effect of the constant power consumption of the air pretreatment system. The UTSD plant typically operates at lower ozone production levels and thus at inefficient power utilization values, because wastewater flows are lower than design and the ozone dosage requirements are not as high as anticipated. Because of the relatively limited information on ozone disinfection, most plants will probably be designed conservatively and therefore will require less ozone than for design conditions for most of the plant life. Consideration should be given during plant design to provide a power efficient unit over the entire system's operating range.

The UTSD ozone system has been intermittently shut down due to excessively high ambient ozone concentrations in and around the plant area. Several modifications were made to prevent ozone leakage including: contact basin covering and sealing, revisions to basin exhaust fan, modifications to basin baffles and scum skimmers, complete ozone repiping from U.P.V.C. to stainless steel piping and installation of new ozone diffusers. An additional modification that has been designed but not yet installed is an off-gas ozone destruct system.

Much conflicting information still exists concerning ozone system design and operation. Two major areas of disagreement concerning the UTSD ozone system were ozone transfer efficiency and the need for an off-gas ozone destruct system. Based on operation experiences at the UTSD plant, the measured ozone transfer efficiency has conformed to ozone/liquid gas transfer theory when good disinfection occurred. The conclusion is that the contact basin is operating as expected, despite the fact that transfer efficiencies are considerably less than values claimed for ozone systems. Ozone in the basin off-gas has existed and has caused ozone exposure to plant operating personnel. To avoid this potentially hazardous situation an off-gas ozone destruct unit was deemed necessary at the UTSD facility and should be strongly considered as part of an ozone wastewater disinfection system.

INTRODUCTION

Background, Purpose and Scope

The Upper Thompson Sanitation District (UTSD) was formed to provide sanitary services to the area surrounding the town of Estes Park, Colorado. Estes Park is located about 75 miles northwest of Denver, Colorado, and is adjacent to Rocky Mountain National Park. The elevation of the community is 2,286 m (7,500 ft). above sea level. The UTSD wastewater treatment .plant discharges to the Big Thompson River which is a cold water fishery and highly visible with respect to the tourist industry. The plant was placed in operation in April 1976, and has a design flow rate of 5,680 cu m/day (1.5 mgd). Significantly large seasonal variations in plant flow occur due to the high influx of tourists in the summer months. The plant flow rate has ranged from 1,140 to 3,790 cu m/day (0.3 to 1.0 mgd). Several unit processes were incorporated into the plant's advanced wastewater treatment (AWT) design to handle the flow variations and discharge a high quality plant effluent. Among these unit processes were flow equalization, activated sludge, attached growth nitrification, tri-media filtration and ozone disinfection.

The incorporation of several unique unit processes and the combination of these processes at one facility aided the UTSD in obtaining funds from the U.S. Environmental Protection Agency (EPA) for a two and one-half year research project entitled, "An Evaluation of Pollution Control Processes — Upper Thompson Sanitation District". Data collected for the research effort included all unit processes. The scope of this paper includes the ozone disinfection process only.

The UTSD ozone system was one of the first full-scale ozone wastewater disinfection processes in the United States. Several state-of-the-art problems were encountered, and continuous ozonation has not yet occurred. As such, limited cost and performance information was developed. This paper discusses the problems encountered with the UTSD ozone system and presents conclusions on various aspects of operation, maintenance and design.

UTSD Plant Description

The UTSD advanced wastewater treatment facility and 33 miles of interceptor and collection lines were constructed concurrently. The majority of flow received at the plant is from domestic and commercial sources. Since the plant discharges to the Big

Thompson River, stringent effluent requirements were established to protect the high quality of the river. (See Table 1).

TABLE 1. PLANT EFFLUENT DESIGN CRITERIA FOR THE UTSD

Parameter	30-Day Average	7-Day Average
5 Day BOD (mg/l)	5	10
Total Suspended Solids (mg/l)	5	10
Fecal Coliform (#/100 ml)	200	400
Ammonia (as N - mg/l)	2.0	—
Chlorine Residual (mg/l)	0.1	0.4
Residual D.O. (mg/l)	2.0	2.0

Figure 1 shows a flow schematic of the processes that were selected to achieve the desired effluent quality. The process design flow is 5,680 cu m/day (1.5 mgd), with a hydraulic capacity of 13,250 cu m/day (3.5 mgd). Parallel treatment systems are provided for the activated sludge and nitrification processes. At flows of 2,840 cu m/day (0.75 mgd) or less, only one-half of these processes are operated.

All flow entering the plant is pumped to an equalization basin. A controlled rate of flow is directed through an aerated grit chamber to the activiated sludge process. Secondary clarifier effluent is pumped to a fixed-film nitrification tower. Tower effluent is directed through multimedia filters to the plant disinfection process. All sludge generated within the plant was wasted to aerobic digesters, concentrated on a pressure roller filter and hauled to a sanitary landfill.

Stringent disinfection and stringent chlorine residual effluent requirements necessitated that alternative disinfection methods be considered. Chlorination plus dechlorination and ozonation were evaluated. It was determined that chlorination plus dechlorination would have required continuing chemical costs and necessitated complicated control equipment to feed and maintain proper dosages. Additionally, dissolved oxygen uptake during dechlorination chemical addition would have required reaeration capabilities prior to effluent discharge. Finally, the hazards and costs of transporting chlorine to the mountain plant site made the chlorination alternative unattractive. With ozone, it was felt that disinfection standards could be maintained, dissolved oxygen would be added to the effluent and the ozone could be generated on-site. Studies had also shown that ozone was effective in color and turbidity reduction, which would be beneficial because of the plant discharge going to the high quality waters of the Big Thompson River. The ozone system was finally selected as the disinfection process.

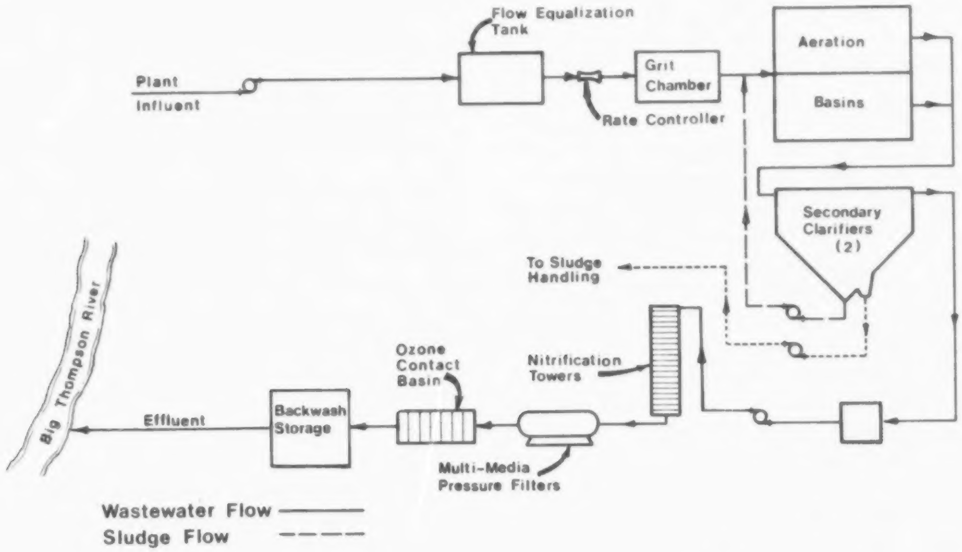

Figure 1. Plant flow schematic diagram for the UTSD.

OZONE SYSTEM DESCRIPTION

The UTSD ozone disinfection process consists of two air-fed ozone generation systems and one ozone contact basin. Each ozone generation system is designed to provide adequate disinfection at the plant design flow rate of 5,680 cu m/day (1.5 mgd) The other generation system was provided for stand-by capacity. The two ozone generation systems were identically constructed and were labeled No. 1 and No. 2. The ozone research data presented in this report is for Generation System No. 2.

Ozone Generation System

At the USTD plant, ozone is generated with Welsbach air-fed units, Model CLP-68F20L. A schematic diagram of the ozone generation equipment is shown in Figure 2. Three components make up each ozone generation system: generator, power supply, and air pretreatment. Each generator was designed to produce 1.43 kg/hr (76 lb/day) of ozone at a minimum concentration of 1% by weight. This provided for a maximum ozone/liquid dosage of 6 mg/l at the plant design flow of 5,680 cu m/day (1.5 mgd). The generators are "iron lung" tube-type units containing 68 discharge chambers. The generators are water cooled using potable well-water at a rate of approximately 1.26 l/sec (20 gpm). Power is supplied to the generator through variable voltage transformers. A controller assembly is used to adjust the voltage from the transformer to the generator, which controls the ozone generator output. The controller was designed for manual or automatic adjustment. During the research project, manual adjustments based on generator amperage readings were used.

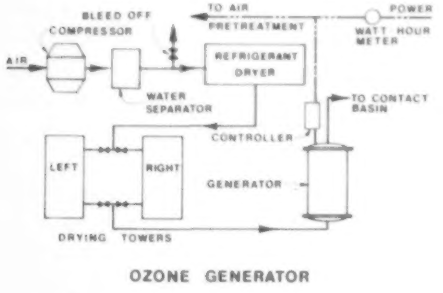

Figure 2. Ozone generation schematic diagram for UTSD.

The air pretreatment components of the ozone generation system are an air compressor, refrigerant drier and air drying towers. Air pretreatment was designed to provide particle-free dry air with a dew point of -51°C at a pressure between 0.42 and 1.05 kg/sq m (6 and 15 psig). During the research project, the air pretreatment pressure was maintained at 0.53 kg/sq m (7.5 psig).

Ambient air is compressed to 0.53 kg/sq m (7.5 psig) by a Nash Model L3 water ring compressor. The compressor operates at a constant speed and has an output of about 160 cu m/hr (94 scfm). It should be noted that standard conditions used throughout this report are one atmosphere pressure and 25°C temperature. A baffle separator is provided to separate the water and air. A bleed-off air valve is provided downstream of the baffle separator. The bleed-off valve at this point was determined to be necessary during the research project to enable the air flow rate to the drying towers to be controlled to prevent overloading of these units.

Compressed air is cooled to between 3.3°C and 5.6°C in a Zeks, Model 9J refrigerant drier, in order to remove excess moisture in the air. The unit was designed so that the air dew point does not exceed 8.9°C.

Refrigerant dried air is further dewatered to a dew point less than -51°C in Kemp Model 100 UEA-1 absorptive drier. The drier uses molecular sieves and activated alumina as absorptive material. Dual towers are provided for continuous operation. Tower operation is cycled at 8-hour intervals to provide regeneration of one tower while the other is in use. Each tower was rated by the manufacturer at 131 cu m/hr (78 scfm) air flow.

Several design modifacations and operation and maintenance changes were implemented based on problems encountered with the air pretreatment and ozone generator units. These problems and the approach to their solutions are further described later.

Ozone Contact Basin

The ozone contact basin is located adjacent to the tri-media filters in the main control building. The ozone generation systems are located directly above the contact basin. The ozone contact basin is 1.30 m (4.25 ft) wide, 11.6 m (38.0 ft) long and 3.66 m (12 ft) deep, which would give an ozone contact time of 14 min at 5,680 cu m/day (1.5 mgd) design flow. The first 8.22 m (27 ft) of the ozone basin is divided into nine equal sized compartments with U. P. V. C. baffles. The baffles are placed to allow vertical, serpentine flow of wastewater through the basin. A schematic of the contact basin is shown in Figure 3.

Treated water from the ozone contact basin passes over a weir and into a backwash water storage basin where it is used for backwashing the tri-media filters. The backwash water storage basin is located beneath the main offices for the UTSD facility. Effluent discharges from the backwash water storage basin through a morning glory overflow weir to the Big Thompson River. The ozone contact basin and the backwash water storage basin share a common air space above their water levels due to the described overflow weir arrangement. Ozone off-gas problems within the main offices occurred because of this arrangement, and modifications were necessary to correct these problems.

The contact basin is covered with aluminum plates which are bolted in place. Hypalon gasket material was placed beneath all joints. Additionally, silicone sealant was used to cover all exposed joints. Ozone laden off-gas from both the contact and backwash water storage basins is vented to the roof of the main control building. The exhaust fan is located in the roof and provides a negative pressure (about 1/4 inch water) in each basin so as to prevent ozone leakage into the main control building and/or offices. A water spray nozzle is located in the vent duct above the ozone basin tank cover to prevent foam from blocking the exhaust air flow.

Ozone is injected into the wastewater in the contact basin through porous stone diffusers. The diffusers are Kullendite, Model FAO 50 as manufactured by Ferro Corporation. The diffusers are located in each of the nine baffled areas. Each diffuser is 6.4 cm (2-1/2 in) in diameter and 61 cm (24 in) long, and has an air permeability between 12 and 15 scfm/ft^2/in. thickness and a maximum pore diameter of 140 microns. Distribution piping consists of type 304 schedule 40 stainless steel piping with welded or threaded joints. Distribution of ozone to each compartment is controlled by nine individual valves. Both the diffusers and distribution piping represent modifications of the original equipment.

Four adjustable height weir scum skimmers are located along the length of the ozone contact basin to facilitate removal of any scum that may be generated as a by-product of ozonation. Scum removed by the skimmers can be pumped to the head of the plant for recycle or to the secondary clarifiers. To date, only small amounts of scum have formed which has been easily handled by the existing scum skimmers.

A proposed ozone off-gas destruct unit is shown in Figure 3. A heat/catalyst ozone destruct unit has been designed and is currently being constructed. The ozone destruct unit represents a major design modification which will be further discussed later.

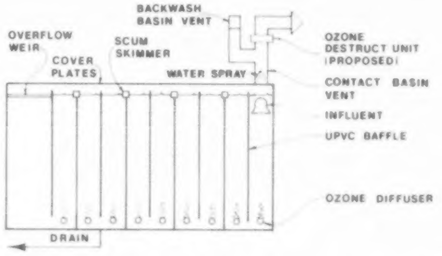

Figure 3. Ozone contact basin schematic diagram for the UTSD.

OZONE SYSTEM DATA COLLECTION

Data collection for the ozone system comprised several different areas: chemical and microbiological analyses, ozone in air concentrations and mass measurements, ozone in water-concentration measurements, electrical power consumption measurements and miscellaneous measurements. In this section of the paper the equipment and procedures used to collect these data are described.

Chemical and Microbiological Analyses

Most chemical analyses of the ozone contact basin wastewater influent and effluent were conducted on samples which were collected at 2-hour intervals, 24-hours per day, 5-days per week (Sunday through Thursday). For some chemical and for all microbiological analyses, grab samples were taken because of the nature of the test. The various chemical and microbiological analyses conducted and the type of sample collected is shown in Table 2. All analyses were conducted in accordance with procedures outlined in *Standard Methods for the Examination of Water and Wastewater, 14th Ed.* (2).

Ozone in Air-Concentration and Mass Measurements

The ozone concentration in the ozone/air flow from the generators was measured by a wet chemistry procedure and by a high concentration continuous reading ozone meter (Dasibi High Concentration Ozone Meter, Model 1003-AH). The wet chemistry method involved a sodium thiosulfate titration of a prepared solution of potassium iodide which had been

TABLE 2. LIST OF CHEMICAL AND MICROBIOLOGICAL ANALYSES CONDUCTED ON THE UTSD OZONE DISINFECTION SYSTEM

Parameter	Frequency	Type of Sample
Total Suspended Solids (TSS)	5/wk	24 hr. Comp.
Chemical Oxygen Demand (COD)	5/wk	24 hr. Comp.
Total Kjeldahl Nitrogen (TKN)	5/wk	24 hr. Comp.
Ammonia Nitrogen (NH_3-N)	5/wk	24 hr. Comp.
Nitrate/Nitrite Nitrogen (NO_3NO_2 - N)	5/wk	24 hr. Comp.
5 Day Biochemical Oxygen Demand (BOD_5)	2/wk	24 hr. Comp.
Total Phosphorus (Total P)	5/wk	24 hr. Comp.
Alkalinity (Alk)	5/wk	24 hr. Comp.
Total Coliform (T.Coli)	5/wk	Grab
Fecal Coliform (F.Coli)	5/wk	Grab
Dissolved Oxygen (D.O.)	7/wk	Grab
pH	5/wk	Grab

exposed to a known volume of the ozone/air flow stream.

An alternate acceptable method of monitoring ozone concentration in air was available after the Dasibi continuous reading ozone meter was properly set up and calibrated. Originally, meter readings did not correlate with wet chemistry results. The problem was isolated to the ozone/air and purge air flow rate to the meter. Lowering and controlling the ozone/air and purge air flow rates to about 2 l/min resulted in consistent meter readings that correlated well with wet chemistry results. A comparison of meter results with wet chemistry results is shown in Table 3 and Figure 4. A very good correlation of results exists. Due to the good correlation between wet chemistry and Dasibi meter results, it was concluded that the Dasibi meter could be used to determine the ozone in air concentrations so that data points could be more readily obtained. However, the Dasibi meter was used to obtain only about 25 percent of the ozone/air concentration measurements used in this report, because the problem with the meter was not corrected until the later part of the research evaluation.

The ozone/air concentration was combined with the generator air flow to obtain the mass of ozone produced. The generator air flow was measured with a Fischer and Porter Series 10A3500 "Flowrator" meter. The recorded flow was corrected to standard pressure and temperature conditions of one atmosphere and 25°C.

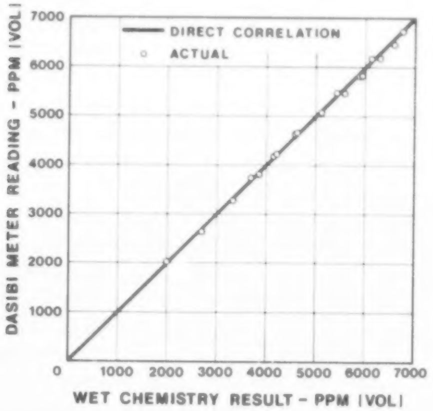

Figure 4. Comparison of Dasibi meter and wet chemistry ozone/air concentration measurements.

TABLE 3. SUMMARY OF COMPARISON OF CONTINUOUS MEASUREMENT DASIBI OZONE METER RESULTS WITH WET CHEMISTRY RESULTS

Date	7/19/78	7/21/78	7/25/78
Dasibi Meter Span Setting	80570	80570	80570
Number of Tests	4	9	5
Average Wet Chemistry* Result [ppm(vol)]	5598	5372	3222
Average Actual Dasibi* Reading [ppm(vol)]	5532	5351	3204
Average Difference (%)**	0.12	0.39	0.56
Range of Difference (%)	-0.17 to 2.0	-0.72 to 2.65	-0.71 to 1.66

* Corrected to standard conditions of 1 atmosphere pressure and 25°C temp.

** Percent Difference = $\frac{\text{(Wet Chemistry - Actual Dasibi) (100)}}{\text{(Wet Chemistry)}}$

The ozone/air concentration of the contact basin off-gas was measured using a wet chemistry procedure. The flow rate of off-gas through the vent duct was measured using a pitot tube. The pitot tube was not ozone resistant so the off-gas flow rate determinations were made with the ozone generator shut down and only the air pretreatment system running. Off-gas flow measurements were taken at different air flow rates from the air pretreatment unit. The data points were very reproducible and a curve was developed relating off-gas flow rate to air flow rate from the ozone generation system. The curve was used to determine off-gas flow rate during testing of the ozone disinfection system. The off-gas flow rate, corrected to standard conditions, was coupled with the off-gas concentration and was used to obtain the mass of ozone contained in the contact basin off-gas.

Ozone In Water - Concentration Measurements

Ozone residual concentrations in the effluent from the ozone contact basin were initially made using a volumetric titration procedure. Using this procedure it became apparent that the color change at the end of the titration was nearly impossible to detect. It was decided that an amperometric titration method would be employed. An amperometric titrator was purchased and was used to obtain ozone/water concentrations. With the present operating procedures, good disinfection results were often obtained with no detectable or negligible ozone residuals.

Electrical Power Consumption Measurements

Ozone generator and air pretreatment power consumption measurements were made with a Sangamo type S3DS watt-hour meter, which was tied into the electrical feed lines to both the ozone generator and air pretreatment units. The meter provides the capability to determine totalized kilowatt-hour readings, maximum kilowatt demand readings and instantaneous kilowatt demand readings, which are the basis of the power cost determination at the UTSD plant.

Miscellaneous Measurements

Other measurements and gauge readings were taken in conjunction with generator production determinations. The air pretreatment dew point was measured with a Shaw Model "S" Mini Hygrometer which had a Red Spot probe. Using the meter, the changes in air dew point from the air pretreatment system were recorded throughout the day. The air dew point typically ranged from -70° C to -54° C.

The air compressor seal water pressure reading was recorded daily and also every time ozone generator production testing was conducted. The seal water pressure was the line pressure of the water that entered the water ring air compressor, and was located before a filter screen in the line. The seal water pressure indicated the relative flow of water to the water ring compressor, in that a higher pressure forced more water to the compressor when the filter screen was clean. Varying seal water pressure resulted in operational problems with the ozone generator, which are described later. A greater or lesser water flow to the compressor was important in that it influenced the compressed air temperature, which in turn affected power consumption of the refrigerant drier and will be discussed later.

RESULTS AND DISCUSSION

The discussion of results from the research evaluation of the UTSD ozone disinfection system are separated into four general categories: ozone generation, ozone system power requirements, ozone contacting system, and disinfection performance.

During the UTSD research evaluation the ozone disinfection system was intermittently operated. Intermittant operation resulted from numerous problems requiring design or operations changes. Several of the problems encountered were associated with the state-of-the-art design of one of the first full-scale ozone wastewater disinfection systems in the United States. Much conflicting and confusing information was provided by various "ozone experts" about the UTSD ozone disinfection system. The conflicting information provided often delayed the correction of the design and operational problems encountered. The major disadvantage of the intermittent operation of the ozone disinfection system is that a thorough evaluation of disinfection performance was not achieved.

Ozone Generation

The three components of the ozone generation system are air pretreatment, ozone generator and power supply. These three areas were evaluated both separately and in combination during the course of the research effort. The ozone generation system performance is dependent upon good quality dry air. As such, the operation and performance of the air pretreatment system is just as important as the operation and performance of the generator itself.

Air Pretreatment. Several problems were encountered with the air pretreatment system, most of which have been corrected. However, the potential for some problems to reoccur exists and studies are in progress to eliminate this potential.

The air compressor for each air pretreatment system is a constant speed unit and continuously discharges an air flow of 160 cu m/hr (94 scfm). By bleeding-off excess air, the air pretreatment system can operate at variable air flow rates. Originally, excess air from the compressor was bled-off *after* passing through the air drying tower. Each tower is rated at a maximum air flow of only 130 cu m/hr (78 scfm), and thus was constantly overloaded. This caused an excessively high dew point of the "dried" air. Subsequently, a minor modification was made by installing a bleed-off valve after the air compressor and before the air drying tower so that excess air was bled-off *before* the air drying tower. This modification solved the operational problems of excess air flow to the drying tower. Variable air flow *not* to exceed the air pretreatment system's ability to provide dry air is highly desirable, but more economical means of providing variable air flow are required and will be discussed later.

Compressed air is directed through a refrigerant drier that has an input voltage of 440 volts. This voltage is compatible with the voltage to the ozone generator, but when operational problems were encountered with the refrigerant drier it was quickly learned that all parts locally available were for 220 volt refrigeration units. These parts were not suitable for the installed 440 volt refrigerant drier, and parts had to be special ordered which delayed the unit's repair. The time delay for repair of the refrigerant drier was the main reason Generation System No. 1 was not operable during most of the time the production data used for this paper was developed. To maintain both ozone generation systems fully operational, the UTSD will have to purchase additional spare parts for the refrigerant driers. Also, the UTSD staff was not trained nor had the equipment to repair the refrigerant unit. The corrective maintenance and spare parts problems encountered with the UTSD 440 volt refrigeration system should be considered in the design of air pretreatment capability for other ozone systems.

Each of the two air drying towers contains activated alumina, molecular sieves and alumina balls desicant material to absorb the water in the air and lower the dew point to less than -51°C. One tower "dries" air from the compressor while the other tower is regenerated by a combination of heating the desicant material to release the bound water and puring the tower contents with dry air to remove the excess moisture. The towers are cycled for drying and regeneration on 8-hour intervals.

Generally, the air drying towers worked well after the air bleed-off valve was moved to a location in the air flow scheme which was prior to air entering the towers. However, a malfunction of the tower switching mechanism used to alternate the towers from the drying to regeneration cycles occurred. This problem occurred due to "sticking" of the linkage of the pneumatically operated switching mechanisms. The arm was lubricated and this problem was corrected, but the potential for this problem to reoccur still exists. The signal for the tower change is electrical while the tower switching mechanism is pneumatic. If problems are encountered with the pneumatic switching mechanism, the electrical system will still indicate that the towers are functioning normally even though "wet" air could be passing through the tower that is regenerating. If this occurred, excessive moisture could be directed to the ozone generator. Under this condition the ozone generator could be "flooded". The term flooded is used to describe moisture build-up in the ozone generator which causes short-circuiting and can cause electrode tube and/or fuse failure.

On several occasions flooding of the ozone generator did occur. On one occasion flooding occurred due to a problem with the refrigerant drier. The refrigerant drier motor overheated and burned out, for an as yet unknown reason. A new refrigerant drier has been ordered. When received, careful electrical checks will be made during its installation to try to isolate the problem.

Flooding of the ozone generator on three other occasions has been tentatively associated with the seal water pressure of the water ring air compressor. These three generator floodings were expensive because several electrode tubes and fuses blew out. As described earlier, the greater the seal water pressure the greater the water flow rate through the compressor and the lower the temperature of compressor air. The temperature of the compressed air is important because if the temperature is too high the refrigerant drier cannot cool the air to reduce the dew point and the air drying tower will be overloaded. The flooding problems were believed to have occurred due to plugging of an in-line filter screen which was used to remove any particulate matter from the water that was directed to the compressor. Plugging caused the flow of water to the compressor to decrease and the temperature of the compressed air to increase, which eventually led to overloading of the drying tower. The entire seal water pressure system is being re-evaluated to determine what design modification or preventive maintenance checks can be instituted to reduce the frequency and/or effects of the screen plugging problem.

Although numerous sources of ozone generator flooding problems have been isolated, the possibility of flooding still exists. Based on this experience, it appears that a high dew point alarm and an associated automatic generator shut-off is necessary to prevent the expense and loss of production associated with generator flooding. Originally, the relative air dew point was monitored with cobaltous chloride pale blue to pink color changing indicator. This indicator is inadequate as an alarm or protection device for the ozone generator for two reasons: 1) someone has to see the color change and shut down the system before flooding occurs and 2) the color change is not sensitive to gradual changes in dew point so potential problems cannot be detected until they are quite far along.

A dew point indicator was purchased as part of the research project to obtain more exact information on air dew point versus generator production. This indicator has greatly aided in the detection of changes in dew point and correction of problems before flooding of the generator occurred, but has not solved the generator flooding problem. Observations of the indicator on a continuous basis is still required. As a better solution, a high level dew point alarm and associated automatic generator shut-down is being considered. To date, this would have saved the UTSD at least $1,600 in electrode tubes and fuses during the past two years of operation.

Ozone Generator Production. Many parameters can influence the rate of ozone production including: power supply, air dew point, ozonated air temperature which is influenced by the volume of air directed through the generator and the ozonated air heat removal capability of the cooling water jacket. These items are not all inclusive, but were selected for analysis because of their potential application to a variety of ozone generation systems.

The UTSD ozone generators were designed to supply ozone at a rate of 1.43 kg/hr (76 lb/day) at an air flow of 118 cu m/hr (70 scfm). The ozone production for Generation System No. 2 only is discussed, because Unit No. 1 was not operational as described earlier. The production rate for different air dew point levels will be presented because the ozone production rate decreased as the dew point of the air increased. This change in production may be a significant operational consideration with respect to continuous, satisfactory wastewater disinfection. The production rate will also be compared to the relative power setting of the generator for two different air flow rates, namely the design air flow rate of 118 cu m/hr (70 scfm) and a lower air flow rate of 79 cu m/hr (47 scfm).

Ozone Production Versus Air Dew Point. The air dew point readings increased proportionately to the drying tower operating time. Typically, soon after a regenerated tower came online and began drying, the air dew point reached its lowest level. As the tower dried more air, the dew point increased. Apparently, as the desicant absorbed and contained more and more moisture, less moisture was absorbed as indicated by the dew point readings. The rate of increase of the dew point was greater at the design air flow rate of 118 cu m/hr (70 scfm) than at the lower air flow rate of 79 cu m/hr (47 scfm). The changes in air dew point for the two air flow rates versus drying tower operating time is shown in Figure 5.

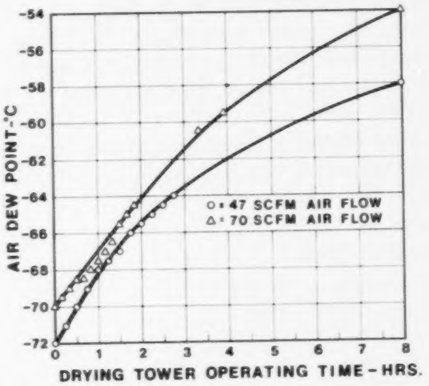

Figure 5. Change in air pretreatment dew point with drying tower operating time (scfm x 1.70 = cu m/hr).

The drying time per operating cycle for each tower was eight (8) hours. The lowest dew point reading shown in Figure 5 is -72° C, although readings as low as -74° C were achieved. The highest dew point recorded was -54° C. All dew point levels recorded, except when obvious air pretreatment problems were noted to cause generator floodings, were better than the manufacturer's rated minimum dew point level of -51° C. In this regard, the UTSD ozone air pretreatment system functioned very satisfactorily.

The change of ozone production per degree of change in dew point was evaluated at different generator power settings. Two power settings were evaluated and results are shown in Figure 6 and in Table 4. In general, the ozone production level decreased as the air dew point increased. It is noted that all the changes in ozone production occurred at better than the manufacturer's rated dew point level of -51° C.

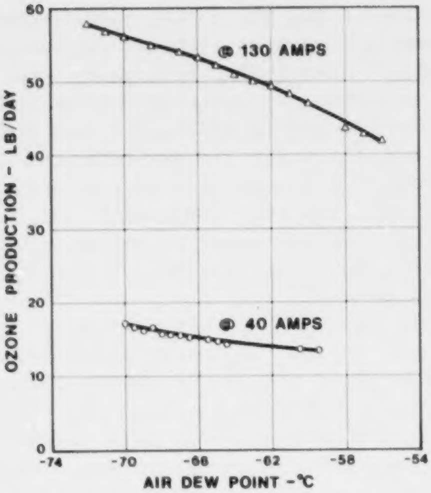

Figure 6. Change in ozone production with dew point at two generator power settings (lb/day x 0.0189 = kg/hr).

As shown in Table 4, the mass decrease in ozone production per degree change in dew point was about three times greater for the 130 amp power setting than for the 40 amp setting. However, the percentage change was slightly less for the 130 amp setting at 1.70%/°C than for the 40 amp setting of 1.93%/°C. From the data presented it was concluded that the overall decrease in ozone production varied considerably with a decrease in dew point, which is important with respect to ozone dosage to the wastewater. These findings would be expected to apply to all ozone generation systems. However, the magnitude of the decrease in ozone production per degree increase in dew point may vary from generator to generator. The implications of these findings on design and operation are significant.

At the UTSD plant the wastewater flow rate was controlled through a flow equalization basin and a negligible variation in daily plant flow occurred. The ozone dosage to the wastewater was manually controlled by adjusting the generator power setting. However, at a given power setting the ozone dosage decreased as the dew point increased. The potential magnitude of the decrease in ozone dosage for observed changes in air dew point is summarized in Table 5.

TABLE 4. EFFECT OF DEW POINT AND POWER SETTING ON OZONE PRODUCTION

	Dew Point	130 Amps	40 Amps
Maximum Production	72°C	57 lb/day	—
	.70°C	—	17.1 lb/day
Minimum Production	56.5°C	42 lb/day	—
	59.5°C	—	13.6 lb/day
Total Dew Point Increase		15.5°C	10.5°C
Ozone Production Decrease		15 lb/day	3.5 lb/day
Specific Ozone Production Decrease			
Mass Decrease		0.97 lb/day /°C	0.33 lb/day /°C
Percent Decrease		1.70%/°C	1.93%/°C

lb/day x 0.0189 = kg/hr

As shown in Table 5, the ozone dosage could vary from 5.0 to 3.7 mg/l if the air dew point changed from -72 to -56.5°C. A change in dosage because of dew point is important from a design and operation basis because disinfection performance is heavily influenced by ozone dosage. Therefore, ozone production information at different power settings and at variable dew point levels are required in order to properly design and operate ozone disinfection systems. In the final analysis it may be required that ozone systems be designed with multiple units which have the air pretreatment drying towers changed sequentially so as to reduce the overall effect of an increase in dew point on ozone production.

TABLE 5. POTENTIAL DECREASE IN OZONE DOSAGE FOR OBSERVED CHANGES IN AIR DEW POINT

	Air Dew Point* -72°C	Air Dew Point* -56.5°C
Wastewater Flow (mgd)	1.37	1.37
Ozone Dosage		
(mg/l)	5.0	3.7
(lb/day)	57**	42**

mgd x 3785 = cu m/day; lb/day x 0.0189 = kg/hr

* It should be noted that the manufacturer's minimum rated dew point level is -51°C.

**For generator power setting at 130 amps

It should be noted that the sensitivity of ozone dosage to system disinfection capability was not evaluated due to the intermittent operation of the UTSD ozone system. It may be that an ozone dosage between 3.7 and 5.0 mg/l yields the same *general* disinfection level, especially when other system variables like influent wastewater COD, TSS and fecal coliform concentrations are considered. If the disinfection capability is not sensitive, the ozone dosage variation because of dew point would not be as critical as earlier described. This aspect should be further evaluated. However, it is still concluded that more information should be developed by ozone manufacturers on ozone production versus dew point levels and ozone production versus generator power settings in order to provide design engineers and plant operators with a better basis for ozone system design and operation.

Overall Ozone Production. The major factors affecting ozone production are generator power setting, air dew point and ozonated air temperature which is influenced by the air flow rate to the ozone generator and the heat removal capability of the cooling water jacket. The cooler the temperature of the ozonated air, the less ozone will be decomposed after it is generated. The UTSD ozone generator begins producing ozone at a consistant, reproducible level at a power setting of 40 amps. The maximum power setting tested was 150 amps, when the generator voltage was 450 volts. For the production evaluation, the mass of ozone produced was determined for power settings at 10 amp intervals between 40 and 150 amps. The ozone production values shown in this paper were taken when the air dew point was between −70°C and −74°C. This dew point level represented the best condition for ozone production within the limits of the air pretreatment unit.

To insure that the cooling water system was performing at optimum conditions, the cooling water jacket was inspected for possible scaling which could have reduced its heat removal effectiveness. No scaling was noted at cooling water jacket sites that were inspected. This was as expected because no scaling problems were encountered on other equipment in the plant that used the same water supply. The temperature of the cooling water ranged between 10° C and 12° C, which was within the ozone manufacturer's specifications.

The actual temperature of the ozonated air was not recorded. However, the relative temperature of the ozonated air was investigated within the limits of the UTSD system by adjusting the air flow rate to the generator. The design air flow rate of 118 cu m/hr (70 scfm) was expected to develop the lowest ozonated air temperature, and in turn produce the highest mass of ozone at a given power setting. The lower air flow rate of 79 cu m/hr (47 scfm) was expected to produce a lower mass of ozone, especially at the higher power settings. The ozone production levels for the two air flow rates is shown in Figure 7. As shown, ozone production is nearly the same for both air flow rates at all generator power settings, although the higher flow rate generally had a slightly higher ozone production level. Apparently, the ozone air temperature change and hence ozone production was not significantly affected within the range of air flow rates tested. The fact that little production difference was shown for the

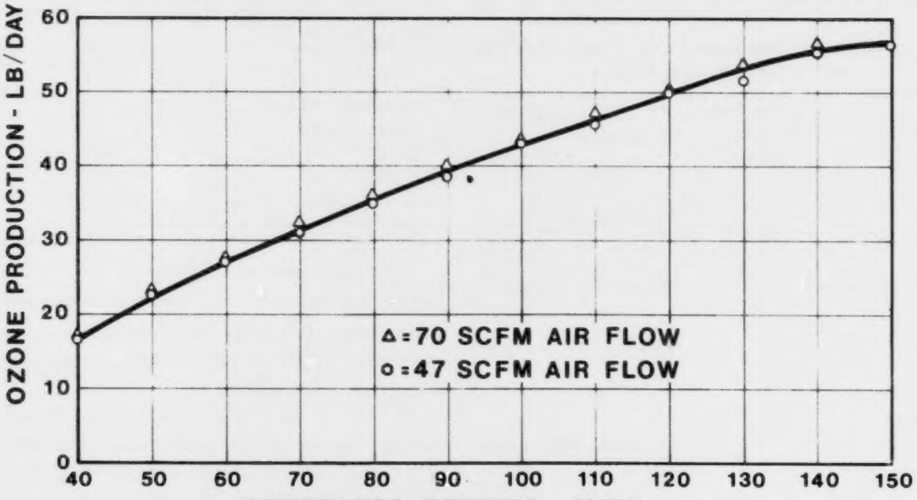

Figure 7. Ozone generator production at various generator power settings (lb/day x 0.0189 = kg/hr); (scfm x 1.70 = cum/hr).

lower air flow rate is significant because a smaller air compressor apparently could have been used which would have resulted in an electrical power savings.

The ozone production levels shown in Figure 7 ranged from 0.32 kg/hr (17 lb/day) to 1.08 kg/hr (57 lb/day), and a fairly uniform increase in production occurred for each 10 amp increase of the power setting. A slight decrease occurred at the highest amp settings, probably due to a relatively higher ozonated air temperature and associated increased destruction of ozone due to temperature. The maximum ozone production level was 1.08 kg/hr (57 lb/day). This occurred at the air flow rate of 79 cu m/hr (47 scfm). The maximum amp setting was not tested at the design air flow rate of 118 cu m/hr (70 scfm), because the ambient ozone concentration within the building became too high on that testing day and the ozone generator was shut down.

The maximum ozone production level of 1.08 kg/hr (57 lb/day) at 79 cu m/hr (47 scfm) air flow was 25 percent less than the manufacturer's rated value of 1.43 kg/hr (76 lb/day). Based on comparable results for other amp settings, it is not expected that the ozone production level at the design air flow of 118 cu m/hr (70 scfm) would be significantly higher. The reason for the lower than design ozone production level was not known and is still being investigated. All known influences on ozone production were optimized during the ozone production tests, including lowest possible air dew point and clean (no scaling) water jacket. Also, prior to production testing the generator was thoroughly cleaned, all electrode tubes were removed and checked for damage, and all tubes were replaced according to the manufacturer's recommendations.

Ozone System Power Requirements

One of the advantages considered in the selection of ozone for the UTSD plant was on-site production. On-site production capabilities were selected over on-going chlorine and dechlorination chemical costs and chemical hauling in the canyon roads which led to the plant. The initial cost of the ozone generation equipment and the on-going power costs needed to generate ozone were considered in this selection. During the research, an evaluation of the ozone generation power requirement was made to determine if the initial cost assumptions were adequate.

The UTSD ozone system has been intermittently operated, and the typical operating procedure was to dose at a rate which would insure disinfection. No attempt was made to optimize ozone dosage. As such, realistic values for on-going power consumption to achieve disinfection were not obtained. However, power consumption of various units over the operating range of the ozone generating system were determined (i.e., ozone generation "mapping"). Each major unit of the ozone system was separately evaluated. Presently, power is consumed by the air pretreatment system, by the cooling and seal water system, and by the ozone generation process. In the future, an ozone destruct unit for the contact basin off-gas will add to the power consumption.

Air Pretreatment and Cooling Water Power Consumption. **Power consumption for the air pretreatment system included power for the air compressor, refrigerant drier, air drying tower heater, pneumatic control system air compressor, and electrical control circuit. The power requirements for the air compressor, refrigerant drier and air drying tower heater were most significant. The power requirement for the pneumatic control system air compressor and electrical control circuit were insignificant in terms of total power usage, and were not included in the power consumption evaluation.**

The air compressor operated continuously and used 8.35 kw of electrical energy. The air compressor was designed to provide a nominal 118 cu m/hr (70 scfm) air flow to the ozone generator. As described, the unit discharged 160 cu m/hr (94 scfm) of air, and the excess air had to be bled-off to avoid overloading the air drying tower. It was also determined that generator production did not significantly change from an air flow rate of 118 cu m/hr (70 scfm) to a lower rate of 79 cu m/hr (47 scfm). It appears that a smaller air compressor could have supported similar ozone production with a significant savings in power consumption, especially at lower than design ozone production requirements.

The instantaneous power consumption of the refrigerant drier was 2.0 kw. However, a lower average daily power consumption was determined because the drier operating time varied with the inlet air temperature and air flow rate to the drier. As described, inlet air temperature increased as the air compressor seal water pressure decreased. Generally, the average inlet air temperature was about 33°C. The relationship between the refrigerant drier operating time and inlet air temperature for an air flow rate of 79 cu m/hr (47 scfm) is shown in Figure 8. At an inlet air temperature of 33°C, the average refrigerant drier operating time was 11.1 hrs/day. A similar evaluation for the higher flow rate of 118 cu m/hr (70 scfm) indicated an average operating time of 14.4 hr/day. These operating times were coupled with the 2.0 kw instantaneous power consumption to determine the average daily power requirements of the refrigerant drier.

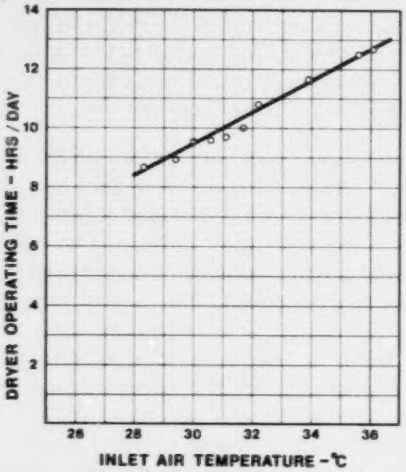

Figure 8. Refrigerant drier operating time at various inlet air temperatures and an air flow rate of 79 cu m/hr (47 scfm).

The air drying tower used electrical energy in the tower regeneration cycle. During regeneration the tower was heated by an electrical heater that had an instantaneous power requirement of 3.65 kw. The 8-hour regeneration cycle consisted of tower "heating" for 4 hours and "cooling" for 4 hours. During the 4-hour heating cycle, heater operation was controlled by a high temperature cut-off and a low temperature start-up system. Therefore, actual heater operating time was less than the total 4 hours. The average heater operating time per heating cycle was determined to be 3.25 hours, resulting in a daily average electrical usage of 1.48 kw.

Air pretreatment power consumption data is shown in Table 6. The daily average power consumption for the air compressor and air drying tower heater was not affected by air flow rate. Power consumption of the refrigerant drier was affected by air flow rate. However, the net effect was that the daily average power consumption was *not* significantly different for the two air flow rates.

One other use of power for the ozone system was supply water for ozone generator cooling and for air compressor operation. This water was provided by the plant potable water pumping system. A special power consumption measurement was taken to determine the power usage of the potable water system, and the average daily power consumption was 2.2 kw. The potable water demand for the ozone generation operation was about 95 percent of the total plant potable water usage. Therefore, the power required to supply water to the ozone generation system was 2.10 kw, and is also shown in Table 6.

TABLE 6. SUMMARY OF POWER CONSUMPTION FOR THE AIR PRETREATMENT AND COOLING WATER UNITS

Unit	Instantaneous Power Consumption	Daily Average Power Consumption	
		@ 47 scfm	@ 70 scfm
	(kw)	(kw)	(kw)
Air Compressor	8.35	8.35	8.35
Refrigerant Drier	2.00	0.93*	1.20*
Air Drying Tower	3.65	1.48**	1.48**
Cooling and Seal Water	5.00	2.10***	2.10***
TOTAL		12.9	13.1

scfm x 1.70 = cu m/hr

* Refrigerant Drier on-time at an average inlet air temperature of 33°C.

** Average drying tower heater on-time of 3.25 hours per 8-hour cycle.

*** Average potable water pump on-time of 10.5 hours. (95% of potable water used for cooling and seal water).

Ozone Generator Power Consumption. The power requirement of the ozone generator increased as the level of ozone production increased. The most important consideration was the power required to produce a given mass of ozone (i.e., power utilization in terms of kwh/lb). Two different air flow rates were used in determining generator power utilization, and power consumption measurements were taken at 10 amp intervals starting where reliable and reproducible ozone production began (40 amps) and were continued to the generator's maximum setting (150 amps). Only Generator No. 2 was used for the power consumption determinations.

Power consumption for the ozone generator had to be carefully determined because power consumption measurements for the air pretreatment and the generator were combined in the readings obtained from the single ozone system watt-hour meter. In order to attain the power consumption for the ozone generator, the power consumed by the air pretreatment units that were *operating at that time* was substracted from the total measured ozone system power that was indicated by the watt-hour meter. Using this procedure, reproducible ozone generator power consumption values were obtained.

TABLE 7. OZONE GENERATOR POWER REQUIREMENTS

	Ozone Production		Ozone Generator Power Consumption		Ozone Generator Power Utilization	
Amperage	@ 47 scfm	@ 70 scfm	@ 47 scfm	@ 70 scfm	@ 47 scfm	@ 70 scfm
(amps)	(lb/day)	(lb/day)	(kw)	(kw)	(kwh/lb)	(kwh/lb)
40	16.8	17.1	4.4	4.6	6.2	6.5
50	22.8	22.9	6.2	6.4	6.5	6.7
60	27.4	——	7.8	——	6.9	——
70	31.5	32.5	9.7	9.9	7.4	7.3
80	35.1	36.2	11.4	11.4	7.8	7.6
90	——	40.0	——	13.1	——	7.9
100	43.1	43.9	15.1	15.1	8.4	8.3
110	45.9	47.2	17.0	17.2	8.9	8.7
120	50.0	50.3	19.7	18.8	9.5	9.0
130	51.8	53.9	21.4	20.8	9.9	9.3
140	55.4	56.8	23.5	22.6	10.2	9.6
150	56.3	——	25.1	——	10.7	——

scfm x 1.70 = cu m/hr; lb/day x 0.0189 = kg/hr; kwh/lb x 2.21 = kwh/kg.

The ozone generator production, power consumption and power utilization values for the two evaluated air flow rates are shown in Table 7. Generator power consumption varied from a low of 4.4 kw to a high of 25.1 kw as the generator amperage setting increased. Ozone production also increased as the amperage increased, but at a lesser rate than power consumption as evidenced by the increase in power utilization. Power utilization increased from a low of 13.7 kwh/kg (6.2 kwh/lb) to a high of 23.6 kwh/kg (10.7 kwh/lb), which is graphically illustrated in Figure 9.

The power utilization values shown in Table 7 for Ozone Generator No. 2 were obtained under conditions that would yield maximum ozone production. These conditions were discussed previously and include: air dew point equal to or less than -70 C, all electrode tubes operational, negligible scaling of the cooling water jacket and a recently cleaned ozone generator. It should be noted that a series of power utilization measurements were taken before Generator No. 2 was cleaned. The uncleaned generator power utilization was an average 15 percent greater than values that are presented for the cleaned generator.

Total Ozone System Power Utilization (Existing System). A summary of the total ozone system power requirements for each air flow rate evaluated is shown in Tables 8 and 9. The power utilization values for the two air flow rates were compared graphically in Figure 9. As shown, power utilization for the two air

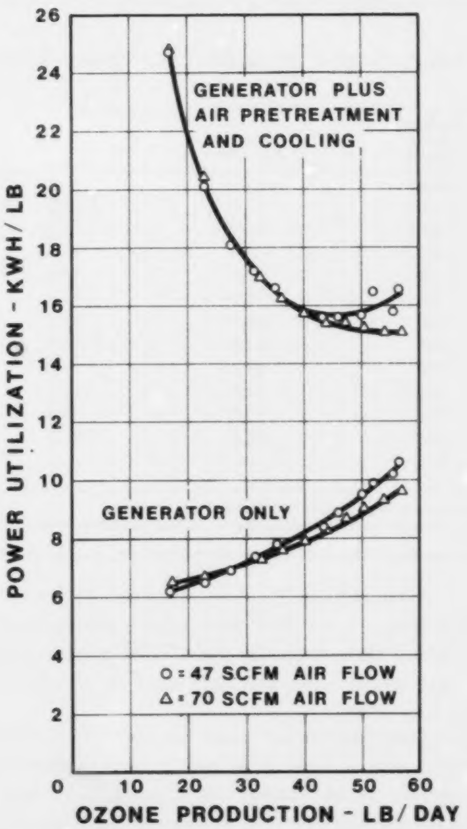

Figure 9. Measured power utilization for the existing UTSD ozone generation system (scfm x 1.70 = cu m/hr; lb/day x 0.0189 = kg/hr; kwh/lb x 2.21 = kwh/kg).

TABLE 8. TOTAL OZONE SYSTEM POWER REQUIREMENT AT AN AIR FLOW OF 79 CU M/HR (47 SCFM)

Generator Power Setting	Ozone Production	Power Consumption			Power Utilization
		Generator	Support Systems	Total	
(amps)	(lb/day)	(kw)	(kw)	(kw)	(kwh/lb)
40	16.8	4.4	12.9	17.3	24.7
50	22.8	6.2	12.9	19.1	20.1
60	27.4	7.8	12.9	20.7	18.1
70	31.5	9.7	12.9	22.6	17.2
80	35.1	11.4	12.9	24.3	16.6
90	——	——	——	——	——
100	43.1	15.1	12.9	28.0	15.6
110	45.9	17.0	12.9	29.9	15.6
120	50.0	19.7	12.9	32.6	15.6
130	51.8	21.4	12.9	34.3	16.5
140	55.4	23.5	12.9	36.4	15.8
150	56.3	25.9	12.9	38.8	16.5

lb/day x 0.0189 = kg/hr; kwh/lb x 2.21 = kwh/kg.
*Air Compressor = 8.35 kw, Refrigerant Drier = 0.93 kw, Air Drying Tower Heater = 1.48 kw and Cooling Water Pumping = 2.10 kw.

TABLE 9. TOTAL OZONE SYSTEM POWER REQUIREMENT AT AN AIR FLOW OF 118 CU M/HR (70 SCFM)

Generator Power Setting	Ozone Production	Power Consumption			Power Utilization
		Generator	Support System	Total	
(amps)	(lb/day)	(kw)	(kw)	(kw)	(kwh/lb)
40	17.1	4.6	13.1	17.7	24.8
50	22.9	6.4	13.1	19.5	20.4
60	——	——	——	——	——
70	32.5	9.9	13.1	23.0	17.0
80	36.2	11.4	13.1	24.5	16.2
90	40.0	13.1	13.1	26.2	15.7
100	43.9	15.5	13.1	28.2	15.4
110	47.2	17.2	13.1	30.3	15.4
120	50.3	18.8	13.1	31.9	15.2
130	53.9	20.8	13.1	33.9	15.1
140	56.8	22.6	13.1	35.7	15.1
150	——	——	——	——	——

lb/day x 0.0189 = kg/hr; kwh/lb x 2.21 = kwh/kg.
*Air Compressor = 8.35, Refrigerant Drier = 1.20 kw, Air Drying Tower Heater = 1.48 kw and Generator Cooling Water = 2.10 kw.

flow rates was not significantly different. Apparently, the difference in air flow rates which represented a fairly broad flow range for the UTSD ozone system was not great enough to effect a change in power utilization.

The power utilization values for the UTSD ozone generator were at least 30 percent greater than typical or expected power utilization figures. The major reason for the higher power utilization was lower than design ozone production rates. The maximum ozone production rate was only about 75 percent of the manufacturer's rated capacity. If the UTSD ozone generator had been achieving its rated ozone production levels at existing power consumption rates, the resulting power utilization values would have been much lower as shown in Figure 10. Efforts to achieve a lower power utilization for the UTSD ozone generators are still being conducted.

The relatively high values for power utilization observed for the UTSD ozone generation system can hopefully be reduced, but it is not expected that a change would occur that would dramatically alter the shape of the total power utilization curve as shown in Figure 10. The shape of the observed and theoretical power utilization curves indicates that dramatically less efficient electrical energy usage occurs at lower ozone production requirements. The fact that the power utilization increases dramatically at low production requirements has significant impact when applied to wastewater treatment plant operation.

At the UTSD plant, variations in wastewater volume and effluent quality occur. Although flow equalization is available, plant flows have ranged from 1140 cu m/day (0.3 mgd) to 3780 cu m/day (1.0 mgd). To optimize ozone dosages in line with these flow variations, an adjustable ozone generation system is required. An adjustable system is available at the UTSD, but low ozone production operating requirements have caused the system to operate in the least efficient electrical energy usage range.

Similarly, municipal wastewater treatment plants are typically designed for flow rates greater than existing rates. Also, because of the limited information available, most ozone systems will probably be conservatively designed with respect to ozone dosage. The combination of these factors will result in lower ozone production requirements than the design capability of the system. At a lower ozone production level, the power utilization value could be much greater than at design production levels which would result in operation of a proportionately less economical ozone disinfection system than would occur at design flows. The need for adjustable ozone production exists, and in order to have this flexibility and be more energy efficient a more uniform power usage efficiency is desireable. Multiple ozone generation and/or multiple air pretreatment units should be considered in order to achieve more uniform power utilization values for the expanded range of ozone production requirements associated with municipal wastewater treatment facilities.

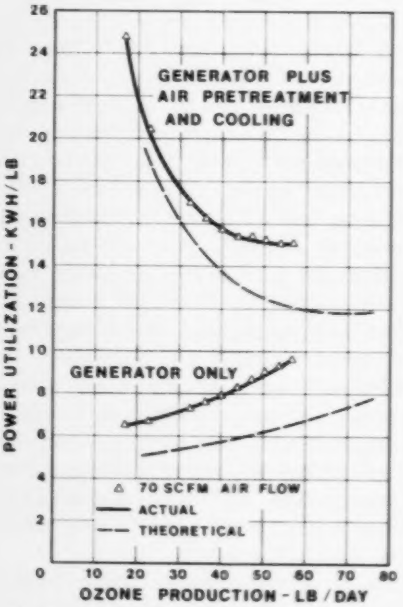

Figure 10. Comparison of theoretical and actual UTSD ozone generation system power utilization (scfm x 1.70 cu m/hr; lb/day x 0.0189 = cu m/hr; kwh/lb x 2.21 = kwh/kg)

Proposed Off-Gas Ozone Destruct Unit Power Requirement. The UTSD ozone system was intermittently operated because of periodically high ambient ozone concentrations in the plant working environment. Several modifications to the ozone piping and contact basin were made to reduce the ambient ozone concentration when the generators were operated. One remaining modification is an ozone destruct unit for the off-gases from the ozone contact basin. The unit has been designed and is being constructed, and represents another source of power consumption associated with the ozone system. The expected power consumption is between 8 and 10 kw.

Ozone Contacting System

Ozone produced in the generators was directed to the ozone contact basin. Several design modifications were made to the ozone contact basin and ozone piping arrangement. Some of the modifications represent state-of-the-art design changes that evolved over the 2-year operation period of the UTSD ozone system. The modifications made include: contact basin covering, basin exhaust changes, baffle and scum skimmer changes, and ozone piping and diffuser replacement. In addition, an ozone destruct system has been designed and is in the process of being constructed.

Covering of Contact Basin. An initial obstacle in operating the ozone disinfection system was the presence of high ambient ozone concentrations in the operator's working environment. Present minimum acceptable standards for human exposure to ozone are 0.1 ppm by volume for a period not to exceed 8 hours. During initial start-up ambient ozone concentrations of 3-5 ppm by volume for 2-hour periods, with peaks of 15-30 ppm by volume were encountered.

It was determined that a portion of these high ambient ozone concentrations were the result of a partially covered contact basin. Additionally, the part of the tank which was covered was not sealed. The contact basin was modified by covering the entire basin with aluminum plates and sealing the joints with hypalon gasket material. It was anticipated that a covered and sealed basin would prevent off-gas leaks.

When the revisions were completed, ozonation was again attempted but high ambient ozone concentrations still occurred. This problem was isolated to ineffective sealing of the basin cover. A silicone caulk was applied in a continuous bead to obtain a positive seal. High ambient ozone concentrations above the contact basin were reduced, but were then detected in the main plant offices. Off-gas that could no longer escape through the basin cover was being forced to the air space above the backwash water storage basin. This basin was adjacent to the ozone contact basin and below the main plant offices. Additionally, contact basin off-gases were not being properly vented to the roof discharge because foam produced by the addition of air and ozone to the contact basin was blocking the exhaust air flow to the vent duct.

Ozone Contact Basin Exhaust System Revision. To correct the problem of off-gas leakage from the backwash storage basin, a duct connecting the existing ozone contact basin with the air space above the backwash storage tank was installed. The addition of the duct eliminated high ambient ozone concentrations in the plant offices, because when foam blocked the movement of air above the water surface of the ozone contact basin the ozone laden off-gases would transfer to the backwash water storage basin where they would be pulled back into the main exhaust duct and discharged. This duct provided an alternate means for ozone laden air to be removed when foaming occurred. In addition to the duct modification, a water spray nozzle was installed in the ozone contact basin duct to depress the foam as it developed. It should be noted that the excessive foaming problem occurred during ozone system start-up and typically lasted for only 2-3 hours.

Baffles and Scum Skimmers Revisions. During initial operation of the ozone system it was determined that short-circuiting of flow was occurring across the top of the compartment baffles. This short-circuiting resulted when air/ozone from the diffusers "air-lifted" the water level in the basin. The baffles were raised slightly, but not to the extent that air movement to the exhaust vent was blocked.

The air-lift action caused by the diffuser system also caused flooding of the scum skimmer mechanisms which were initially set too low. The adjustment range of the skimmer units was expanded and adjustment handles were extended through the basin cover. To facilitate adjustment of the overflow weir plates, plexiglass sight windows were installed into the basin cover above each skimming unit. To date, the continuous use of the scum skimmers has been unnecessary as little foam or the predicted gelatinous type froth has been produced. The lack of appreciable foam or froth is believed to be due to the high quality water entering the ozone contact basin. This water has very little material (i.e., total suspended solids) available to be coagulated into froth.

Ozone Piping Modifications. The combination of modifications previously discussed allowed operation of the ozone system whenever the wind was blowing sufficiently to remove the ozone laden off-gases from the area surrounding the discharge stack. The system had to be shut down when the wind was not blowing because high ambient ozone concentrations would develop in and around the plant area. Plans were developed to install an ozone destruct unit for the contact basin off-gases to prevent excessive discharge of ozone to the atmosphere. The UTSD was instructed by State Department of Health officials to operate the ozone system to achieve disinfection, but the District was allowed to shut down the system when necessary to prevent human exposure to high ambient concentrations of ozone. Under this arrangement, it is estimated that the ozone disinfection system was operated only about 50 percent of the time.

The UTSD ozone system was operating under these conditions for about one year when problems were encountered with leaks in the ozone piping from the generator to the ozone diffusers. The original ozone piping from the generator to the ozone diffusers was Schedule 40, U.P.V.C. pipe with both solvent weld and threaded joints. The problems with leakage occurred around the joints and along straight pipe sections near pipe hangers. This leakage problem may have been caused by inadequate care during installation; however, good plumbing practices by the contractor were felt to exist because very few problems occurred with other plant piping systems. Although no definite conclusions could be developed, the leakage in the ozone piping *may have* occurred as a result of ozone exposure over a one-year time period and *not* due to poor workmanship.

Because of the excessive ozone leakage of the U.P.V.C. pipe, all piping was replaced with Schedule 40, Type 304 stainless steel. Some of the stainless steel pipe connections were threaded and some were welded. Some of the threaded stainless steel connections were noted to leak and were tightened, but some could not be sealed and were welded. It was concluded that welded connections provided the best assurance for sealing ozone piping.

The U.P.V.C. piping within the ozone contact basin was also replaced with stainless steel, primarily to facilitate installation of new ozone diffusers that were concurrently installed. The diffusers were connected to the stainless steel pipe outside the basin, and the pipe with diffusers was lowered into the basin aided by the structural integrity of the stainless steel pipe. During installation another reason for using stainless steel pipe became evident. Upon removal of the U.P.V.C. pipe it was observed that the portion of the pipe that had been underwater was extremely brittle and shattered easily when dropped. The U.P.V.C. pipe's condition was not felt to be acceptable for long-term operation and replacement with stainless steel pipe was considered appropriate. These results indicate that strong consideration should be given to using suitable grade stainless steel pipe and welded connections for all ozone/air piping.

Ozone Diffuser Modifications. The UTSD ozone contact basin achieved ozone transfer efficiencies ranging from 35 to 70 percent. Transfer efficiency (TE) as used in this context was calculated as follows:

$$\text{TE} = \frac{(\text{Mass of Ozone Produced - Mass of Ozone in Off-Gas})(100)}{\text{Mass of Ozone Produced}}$$

Typically, the transfer efficiencies were between 50 and 60 percent, which were considerably less than the design TE of 90 percent. One reason for the lower than expected TE was breakdown of the ozone diffusers. The original ozone diffusers were tubuler in shape and were attached to a piping connection nipple with a "2-part epoxy bond." The epoxy served in a structural as well as a gas sealing capacity. Because the measured transfer efficiencies were lower than expected, the contact basin was drawn down and the diffusers were inspected. It was noted that the epoxy had become extremely soft. The diffuser manufacturer claimed that the epoxy would probably become "a little soft" when exposed to water. A new diffuser was exposed to water for 3 months in the laboratory. No softening effect was noted. During this time the TE had reduced to only about 35 percent from 50 to 60 percent. The contact basin was again drained and this time some diffusers were noted to have completely separated from the connection nipple and had fallen to the bottom of the contact basin.

The original ozone diffusers were replaced with new diffusers at the same time the stainless steel piping modification was completed. The new ozone diffusers were of similar material and were also tubular in shape, but a bolted stainless steel connection and hypalon gasket material were incorporated in the diffuser construction. The reason for replacing the original diffusers was to obtain a diffuser that hopefully was more ozone resistant. Initial indications are favorable, but operation with the new diffusers has been limited and final conclusions concerning the suitability of the diffusers has not been reached.

Evaluation of Transfer Efficiencies. After installation of the new ozone diffusers and stainless steel piping, TE tests were conducted and were found to be similar to efficiencies that were attained when the ozone system was first started up, namely 50 to 60 percent. These values were better than the 35 percent efficiency attained when some of the original diffusers were known to have failed, but were still less than the design efficiency of 90 percent. A re-evaluation of the ozone contact basin design was made.

The UTSD ozone contact basin design was similar to a basin that was designed and tested in 1971 at the Louisville, Kentucky Wastewater Treatment Plant. The ozone basin at Louisville was reported to have consistently achieved 90% or greater TE. At about that same time at the University of Louisville, research work was conducted to measure the TE of a contact basin similar to the basin that was tested at the Louisville plant, and a paper was published concerning the results (1). The author reported that the 90% to 95% TE that was achieved at the Louisville plant could not be duplicated at the University laboratory. Laboratory results indicated a TE of about 50%. The difference in transfer efficiency was attributed to a "with reaction" consumptive use of the applied ozone at the Louisville plant, which was ozonating secondary effluent. The tests completed at the University of Louisville, which did not correlate with Louisville plant results, were labeled "without reaction" test results. These "without reaction" tests also correlated well with ozone/liquid gas transfer theories.

Transfer efficiencies achieved at the UTSD facility have also correlated well with ozone/liquid gas transfer theories, and are believed to be more in line with the "without reaction" tests conducted at the University of Louisville. The wastewater ozonated at the UTSD plant is tertiary effluent and of considerably better quality than typically associated with secondary treatment.

The belief that the UTSD ozone contact basin was performing similarly to the "without reaction" tests as established at the University of Louisville was further supported. Periodically, the ozone transfer efficiency has reached a level of about 70 percent. When this occurred, poor disinfection results were often achieved even if disinfection previously occurred at similar ozone dosages. At the same time, the TSS concentration through the basin increased. This condition existed when the ozone generator was started after being shut down for a day or two. The air pretreatment system was always operated; thus air was continuously diffused into the wastewater. When the ozone generator was shut off or was operated at a very low ozone dosage level, biological growth developed in the basin in the form of a slime on the basin walls, the U.P.V.C. baffles, and other surface media in the basin. When the ozone generator was started and operated at a higher ozone dosage, a "with reaction" ozone consumption probably occurred and increased the ozone transfer efficiency to near 70%. At the same time, the biological slime would slough-off and increase the contact basin effluent TSS concentration which interferred with the disinfection capability of the system. However, when the UTSD ozone contact basin is operated on a continuous basis it is expected that it will operate according to gas/liquid transfer theories. It was concluded that the basin is achieving expected ozone transfer efficiency for the quality of wastewater treated. Based on these developments, engineers should design ozone contact basin transfer efficiencies based on ozone/liquid gas transfer theories and not on pilot scale tests.

Biological slime build-up occurs in the UTSD ozone contact basin when the ozone generator is shut down and/or operated at a very low ozone dosage level. Because of this problem with intermittent ozone operation, a good microorganism reduction versus ozone dosage relationship was not obtained. When continuous ozonation and continuous good disinfection is achieved, the ozone dosage will be adjusted to determine the minimum level necessary to achieve disinfection. Transfer efficiency tests will be taken to determine the effective ozone dosage as opposed to applied ozone dosages (i.e., dosage excluding the ozone lost in the off-gas) so that a common ground comparision can be made with other systems that achieve a better transfer efficiency.

Off-Gas Ozone Destruct System. Intermittent ozone generator operation occurs because of periodic, excessive ozone exposure to operating personnel. Originally, the ozone exposure was due to several sources: contact basin leaks, backwash storage basin leaks, ozone piping leaks and excessive ozone in the contact basin off-gas exhaust. Excessive ozone from these sources, except contact basin off-gas exhaust, have been corrected. Ozone operation is now dependent upon wind direction and velocity to appropriately disperse the contact basin off-gas exhaust. To achieve continuous ozonation, several options were considered to control the off-gas ozone discharge. Among these were: heat destruct, heat/catalyst destruct, activated carbon, recycle to sludge and discharge through a tall stack. The option selected was heat/catalyst destruct. Heat destruct was rejected because of an excessively high power consumption. Activated carbon was rejected because of its explosive potential when combined with ozone. Recycle to sludge and discharge through a tall stack were rejected because they were felt to likely result in transferring the problem to another area within the plant. The heat/catalyst ozone destruct system for the contact basin off-gases has been designed and is being constructed. The system is manufactured by Emery Industries. Cost, performance and operating information of the heat/catalyst unit should be developed when installed and operational. An off-gas ozone destruct unit should be strongly considered for all newly designed ozone systems.

Disinfection Performance

Operation of the ozone system at the UTSD plant was sporadic due to a variety of problems that resulted in high ambient ozone concentrations that represented a hazard to operating personnel. Shown below is a synopsis of the problems encountered.

Time Period	Comment
June — December, 1976	Off-gas problems due to contact basin cover problem.
December, 1976 — February, 1977	Good disinfection achieved, but equipment to measure ozone concentration not available.
March — April, 1977	Good disinfection achieved.
May — June, 1977	Poor disinfection achieved because of ozone diffuser problem.
July — October, 1977	System shut down for inspection and repair of original ozone diffusers.
October — December, 1977	Relatively good disinfection, but only with extremely high ozone dosages because of further problems with ozone diffusers.
January — April, 1978	Design, construction and installation of new ozone diffusers completed.
May — September, 1978	Sporadic operation due to excessively high ozone levels in and around the plant area due to no off-gas ozone destruct system.

Routine collection of analytical data for the ozone system was initiated during the week of December 12, 1976. In general, good disinfection performance could be achieved when the ozone diffusers were in good condition and the system was operated for an extended period of time (several days). Disinfection performance was poor when problems were occuring with diffusers and when the ozone system was operated on an intermittent basis. A summary of performance data for selected time periods is shown is Table 10. Periods 1, 2 and 3 represent data collected when the original ozone diffusers were in operation. Period 4 represents data collected after the new diffusers were installed.

During the 8-weeks of Period 1, the original ozone diffusers were new and were operating satisfactorily. Very good disinfection occurred at an average ozone dosage of about 11 mg/l. (Note: Effective dosage was only 50 to 60 percent of the applied dosage.) The effluent fecal coliform concentration was reduced to 30 per 100 ml, much better than the design standard of 200 per 100 ml. The COD reduction during Period 1 was 11.7 percent and the TSS reduction was 35.6 percent. Both the influent COD and TSS concentrations were relatively low at 29.7 and 4.5 mg/l, respectively.

During Period 2, the average applied ozone dosage was about 9 mg/l, but disinfection performance deteriorated significantly. The effluent fecal coliform concentration was 2,080 per 100 ml. The reason for the poor performance was primarily attributed to problems with the ozone diffusers. During Period 3 disinfection improved, but only after the applied dosage was more than doubled to about 19 mg/l.

The performance data for Period 4 represents information collected after the new ozone diffusers were installed. The data was collected for two different time periods because of problems with intermittent ozone generation and biological slime build-up. As during Period 1, good disinfection was achieved during Period 4. The effluent fecal coliform concentration was only 9 per 100 ml at an applied ozone dosage of about 7 mg/l. (Note: Effective dosage was still only 50 to 60 percent of the applied dosage.)

These performance data are limited, but indicate that good disinfection can be achieved with the UTSD ozone system. More definitive conclusions are expected when the ozone destruct unit for the contact basin off-gas is operational and continuous ozonation can be implemented.

TABLE 10. SUMMARY OF PERFORMANCE DATA FOR SELECTED TIME PERIODS

Period	Date	No. of Wks.	Applied Ozone Dosage*	Effluent Fecal Coliform	COD in Effluent & Reduction		TSS in Effluent & Reduction	
			(mg/l)	(#/100 ml)	(mg/l)	(%)	(mg/l)	(%)
1	2/27/77 To 4/23/77	8	11.*	30	26.2	11.7	2.9	35.6
2	4/23/77 To 6/18/77	8	9.**	2,080	39.5	7.3	6.0	22.1
3	10/16/77 To 12/10/77	8	19.**	91	43.3	7.5	7.6	8.6
4	4/16/78 To 4/29/78 Plus 5/21/78 To 6/3/78	4	7.*	9	35.6	10.2	3.1	3.0

* During these time periods the system achieved a maximum of 50 to 60 percent ozone transfer efficiency.
**During these time periods the system achieved a maximum of about 35 percent ozone transfer efficiency.

CONCLUSIONS AND RECOMMENDATIONS

1. The "Dasibi Meter" continuous measurement ozone concentration meter readings correlated well with wet chemistry results after the meter was properly set up and calibrated.

a. Ozone and purge air flow to the meter was controlled at 2 l/min.

b. The meter span setting was adjusted based on wet chemistry results.

2. The dew point of air from the ozone system air pretreatment unit varied, but as a whole the unit performed better than rated by the manufacturer.

a. The rated minimum air dew point was −51°C.

b. The operating dew point ranged from −74°C to −54°C over an 8-hour operating cycle.

3. The ozone generator periodically "flooded" and was damaged due to malfunctions to the air pretreatment system.

a. The cobaltous chloride color changing indicator provided to show an increase in air dew point was not sensitive to gradual changes and potential problems associated with dew point could not be detected until they were quite far along.

b. A dew point measuring device was sensitive to gradual changes in dew point, but "flooding" still occurred when the meter was monitored only once per day.

c. A dew point high level alarm and/or automatic system shut-down would substantially reduce generator "flooding" potential and should be incorporated in all air pretreatment unit designs.

4. The air pretreatment refrigerated drier required special maintenance considerations.

a. The drier voltage of 440 volts was compatible with the voltage of the ozone generator, but required special order parts because 220 volt refrigeration units were more common in the community.

b. Repairs to the refrigerant drier were quite technical and had to be completed by an experienced repairman who had special equipment.

5. Ozone production decreased with an increase in air dew point. (Note: Actual air dew point was better than rated by the manufacturer.)

a. The ozone production decreased by 1.70 percent per °C change in air dew point at a higher ozone production level, and by 1.93 percent per °C change at a lower ozone production level.

b. A decrease in ozone production because of an increase in dew point could decrease the ozone dosage to the wastewater from 5.0 mg/l to 3.7 mg/l.

c. Ozone system air pretreatment units may have to be designed with multiple components that are changed in sequence so as to reduce the overall effect of an increase in air dew point on ozone production.

d. More information should be developed by ozone manufacturers on ozone production versus dew point levels over the entire ozone generator operation range in order to provide design engineers and plant operators with a better basis for ozone system design and operation.

6. Ozone production of the UTSD ozone generator was 25 percent less than rated by the manufacturer, even though production testing was completed under controlled conditions as follows:

a. The air dew point was equal to or less than −70°C.

b. The generator cooling water jacket was free of scaling.

c. The generator cooling water temperature was 10°C to 12°C, which was within the ozone manufacturer's specifications.

d. All electrode tubes were removed, inspected, cleaned and replaced according to the manufacturer's recommendations.

7. Ozone production of the UTSD ozone generators was not significantly lower at air flow rates of 79 cu m/hr (47 scfm) as opposed to air flow rates of 118 cu m/hr (70 scfm).

8. Ozone generation system power utilization at the UTSD plant was greater at lower ozone production levels due to the relatively constant power requirements of the air pretreatment unit, and caused inefficient power usage under existing operating conditions.

a. Total power utilization varied from about 55 kwh/kg (25 kwh/lb) at production levels of 0.32 kg/hr (17 lb/day) to 33 kwh/kg (15 kwh/lb) at production levels of 1.08 kg/hr (57 lb/day).

b. Lower ozone production levels of 0.32 kg/hr (17 lb/day) are usually sufficient at the UTSD plant for current wastewater flow rates and ozone dosage requirements. Thus, the least efficient power utilization values presently exist.

c. More efficient power utilization values over the entire range of ozone generation system operation would improve system power consumption and cost at lower than design wastewater flow rates and ozone dosage requirements, and should be considered for all ozone system designs.

9. Contact basin off-gas, ozone discharge and other sources of ozone leakage have caused excessively high ambient ozone concentrations in and around the plant area and have required that several system design modifications be made.

a. The contact basin had to be totally covered and sealed.

b. The off-gas exhaust system was redesigned.

c. The basin baffles and scum skimmers were modified.

d. The UPVC ozone piping was replaced with stainless steel piping.

e. The off-gas ozone must be destroyed. A heat/catalyst ozone destruct unit has been designed and is being constructed.

10. The original ozone diffusers were not ozone resistant. They failed and were replaced with new diffusers.

11. The UTSD ozone contact basin was designed for 90 percent transfer efficiency and was based on incomplete information.

a. Ozone transfer efficiency was variable and was affected by wastewater quality.

1. Typically, the transfer efficiency was between 50 and 60 percent.
2. The transfer efficiency increased when the wastewater quality was poor.
3. With good wastewater quality, measured ozone transfer efficiencies correlated well with ozone/liquid gas transfer theory.

b. To insure achieving a desired, minimum transfer efficiency, ozone contact basin design should be based on ozone/liquid gas transfer theory.

12. Good disinfection performance has occurred at the UTSD plant when the ozone diffusers were in good condition and the system was operated consecutively for several days.

a. Effluent fecal coliform concentrations averaged less than the design requirement of 200 per 100 ml at applied ozone dosages of about 7 mg/l. (Note: Transfer efficiency was 50 to 60 percent.)

b. When the ozone system is operated for only a short period of time, poor disinfection is experienced and increased effluent TSS concentrations occur due to biological slime growth sloughing off.

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Lab Chemist - Mrs. Sue Martin
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DISCUSSION:

MR. DALE WOOD: I would like to ask, number one, what was the flow capacity of that plant?

MR. RAKNESS: Design flow is 1.5 mgd, operating flow ranges from 0.3 to 1.0 mgd.

MR. WOOD: Does this take into account a flood peak? In this area we have plants that will have a flood condition that is above peak flow during heavy rainfall.

MR. RAKNESS: The maximum hydraulic capacity through the plant is 3.5 mgd, but the 1.5 mgd is the average daily operating design flow for about twenty years from now.

MR. WOOD: Is the capability of your equipment at that level?

MR. RAKNESS; The capability of the equipment is for 1.5 mgd.

MR. WOOD: And is that a single unit or dual units?

MR. RAKNESS: Good question. We have dual units. One unit is operated as a standby. Each unit is capable of 76 pounds per day, which gives a design dosage of 6 mg/l at a flow rate of 1.5 mgd.

DR. RICE: Quick comment in terms of your remark about the uncovered contactors, which had to be covered. Any ozone generator manufacturer that would recommend uncovered contactors should be run out of the business and never talked to again. That cannot be considered a proper design.

MR. WHITE: I think this transfer efficiency is very important. You went by it so fast you lost me; I understand the 90% transfer, which means from what I know about it that 10% comes in the off-gas. But, you were showing 50 and 60% transfer efficiency. Will you explain it again?

MR. RAKNESS: We measured transfer efficiency. We measured the mass of ozone supplied to the ozone contact basin, and the mass of ozone in the off-gas, subtracted those two and divided by the mass of ozone supplied, and that is what we call transfer efficiency.

MR. WHITE: Well, when you showed 50 to 60% transfer efficiency, are you saying that 40 to 50% was coming in the off-gas?

MR. RAKNESS: That is right.

MR. WHITE: Okay.

QUESTION: Instead of running a destruct set up on your off gas, you are certainly throwing away some very valuable material. Why not run a recycle line and either go into a tower feeding the waste stream before your initial diffuser contactors, which certainly most chemical engineering handbooks would show you how to design that, or go to a second bank of diffusers before you go into any destruct mechanism. That is really throwing good dollars away.

MR. RAKNESS: We investigated different ways of handling that off-gas. Among them were heat-only destruct, which was not cost effective at all. Another way was diffusion into the sludge. Another one was diffusion into the wastewater stream. As a result of off-gas that has been in and around the plant area and to which operators were exposed, the operators, the superintendent, the board members, and we did not feel that we wanted to chance transferring the problem to another area. For that reason the last two alternatives were excluded. As far as redesigning the ozone contact basin or adding to it with a diffusion system prior to ozone diffusers, when we considered the cost effectiveness of that, it was just better to destruct. The problem with Upper Thompson (I should not say it is a problem; I guess it is a blessing in disguise) is that we are not at the maximum capacity of the generators. We still have excess capacity. So we have not had to reach the limits.

DR. NOACK: I have just a brief comment on the ozone and the off-gases. I think they will require much attention in days to come in view of the fact that OSHA under their generic carcinogen policy in one of the lists that are attached to the regulatory proposal that classified ozone, I believe it is categorized as a category two carcinogen. I have reviewed the list and I have not found chlorine in there.

MR. ELLNER: I would be curious to know what you found the half life of ozone to be. Our experience has shown it to be rather short, and I am curious why you would be looking for a destruct requirement for the off-gases.

MR. RAKNESS: Well, I should say another alternative that we thought about was just venting it to the atmosphere through a thirty or forty foot stack and let the air take care of it. Right now it is vented on top of the roof about twenty feet above the plant area with operators exposed to it, and that has not been sufficient to disperse the ozone in the atmosphere. Thus, because of aesthetic reasons for a higher tower we did not want to go any higher than that. Also, because it is twenty feet in the air right now, we are not sure that it would be adequately dispersed if we went higher. I am sure that at some elevation you could let it go and it would not affect the ground, but it still would not be aesthetically pleasing to the area.

MR. ELLNER: If you do need a destruct system, another characteristic of 254 nm UV is that it is destructive to ozone.

21.

FIELD-SCALE EVALUATION OF WASTEWATER DISINFECTION BY OZONE GENERATED FROM OXYGEN

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INTRODUCTION

This paper pertains to the findings of a full-scale U.S. Environmental Protection Agency (USEPA) funded R&D study being conducted on the Meander Ozonation System (MOS). The MOS provides the final treatment step of disinfection in the 1.5 × 10^4 m^3/d (4 mgd) regional Meander Advanced Wastewater Treatment Facility (Meander AWTF). The facility is owned by Mahoning County and is located in Trumbull County, Ohio. The entire Meander AWTF, including the ozonation system, was designed by Engineering-Science, Ltd., Cleveland, Ohio.

The effluent discharge standards for the Meander AWTF require summer/winter nitrification, phosphate removal, and postaeration as shown in Table 1. To meet these standards, the following treatment scheme was selected for the Meander AWTF. The wastewater from interceptor is pumped to comminution and aerated grit removal facilities. Carbonaceous BOD is removed in a first-step pure oxygen activated sludge reactor and clarifier, and nitrogenous BOD is removed in a second-step pure oxygen activated sludge reactor and clarifier. Lime is mixed with second-step effluent, flocculated, and the chemical/phosphorus sludge is separated in a solids contact clarifier. Following recarbonation and dual media filtration, the effluent is ozonated for disinfection prior to discharge to Meander Creek. The process flow diagram of the Meander AWTF, including sludge handling facility, is shown in Figure 1.

The USEPA was interested in technical developments which could be forthcoming from the

TABLE 1. OHIO ENVIRONMENTAL PROTECTION AGENCY EFFLUENT DISCHARGE STANDARDS FOR MUNICIPAL WASTEWATERS DISCHARGING TO THE MAHONING RIVER AND ITS TRIBUTARIES

Critical Constituent	Meander Report Design Standard 1970	% Reduction	Present Effluent Standards Effluent Concentration (mg/l)	
	% Reduction	Monthly Average	Monthly Average	Maximum
BOD_5	90	95	10	15
Suspended Solids	95	95	12	18
Free Ammonia				
Summer	—	85	2.5	5.0
Winter	—	85	2.5	5.0
Fecal Coliforms				
Summer	—	99.99	200*	400*
Winter	—	99.99	1,000*	2,000*
Oils and Greases	—	90	10	20
Phosphorus	90	90	1.0	1.5
DO	—		Effluent Concentration 5 mg/l	

* Counts per 100 ml.

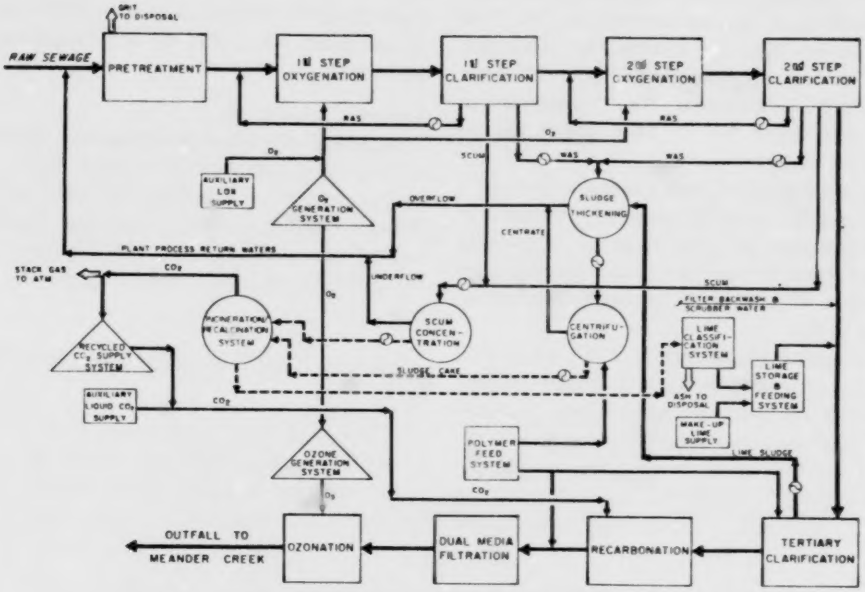

Figure 1. Process Flow Diagram for Meander Advanced Wastewater Treatment Facility

MOS. This facility was considered ideal for a USEPA Research and Demonstration Grant because it was to be the first full-scale plant in the country using ozone to disinfect the final effluent of a wastewater plant, coupled with on-site oxygen generation unit and two-step pure oxygen system. Moreover, the size of the MOS was considered ideal to develop parameters which could easily be scaled up for larger treatment facilities. The USEPA interest in the project finally resulted in a grant to the Mahoning County in June 1976 to conduct a two-year full-scale study on the MOS. Mahoning County is presently conducting this study through its consultant, Engineering-Science, Ltd., Cleveland, Ohio.

The Meander Ozone R&D study was to be completed by 31 August 1978. However, this was not achieved for various reasons resulting from the unique features of the ozonation system. This is the first facility in the country where ozone is produced for wastewater disinfection from oxygen recycled from the contactor. At the time this paper was committed for the symposium, it was expected that the ozonation system would have been operating for several months before the presentation and that there would be sufficient data to report. For several reasons beyond the control of Mahoning County, the ozonation system did not become operational until only a few weeks ago and, therefore, very limited data are available. Under these circumstances, this paper will report on the problems encountered in placing this system into operation, and will report on the limited data obtained as of 31 August 1978.

MEANDER OZONATION SYSTEM

Process Description

The Meander Ozonation System is designed to be operated on air or oxygen recycled from the contactor. The ozonators are capable of producing 180 kg/d (400 lb/d) ozone providing an ozone dosage of 6 mg/l for the peak flow of 3 x 10^4 m^3/d (8 mgd). There are two trains of ozone generation units which can be operated separately or in combinations. Both trains combined can handle flows up to 3 x 10^4 m^3/d (8 mgd) at an ozone dosage of 6 mg/l. However, one ozone decomposer at a time is used since each decomposer is designed to take the full load; thus, one decomposer is at 100 percent standby. A common header between compressor and dryer allows compressor No. 1 to work with dryer No. 2. Electrically, however, compressor No. 1 operates in conjunction with dryer No. 1. Similarly, compressor No. 2 operates in conjunction with dryer No. 2. This system can be operated in two modes.

The first mode is the open-loop system wherein air is used as feed gas for ozone generation, as shown in Figure 2. In this case off-gases from the contacting tank are discharged directly to the atmosphere after residual ozone is destroyed through the ozone decomposing device.

The second mode of operation of the ozonation system is the closed-loop system which utilizes oxygen-enriched feed gas as shown in Figure 3. In this case, the off-gases from the ozone contacting tank are recycled and serve as feed to the ozone generator. Make-up oxygen is necessary in this case to compensate for the oxygen losses in the system.

To handle a broad range of flow variations in the Meander AWTF, a contacting system consisting of eight Positive Pressure Injectors (PPI), each capable of handling 3.8 x 10^3 m^3/d (1 mgd) flow, is also provided.

System Components

In general, the Meander Ozonation System consists of five main components: on-site ozone generation facility, ozone contacting system, ozone decomposing device, control system, and the indicating and recording instruments. Particulars of these components are summarized in Table 2.

MEANDER OZONE R&D STUDY OBJECTIVES

Field Study

The initial field study work is directed towards collecting background data on the plant flow and processes, and to calibrate the instrumentation and perform the mapping of the ozone generating

TABLE 2. MAIN COMPONENTS OF OZONATION SYSTEM AT MEANDER

	No. of Units	Type	Capacity
Compressor	2	Nash Model L-3	Each 142 cfm at 15 psig Outlet Pressure (0 psig Inlet and 60° F)
Dryer	2	Deltech Model ESL-4 (Refrigerative/Dessicant Type)	Each 140 scfm at 15 psig
Ozone Generator	2	Union Carbide Model A-128 Lowther Plate 9,000 volts (peak) 2,000 Hertz	Each 100 lb/day (Air) 200 lb/day (Oxygen)
Ozone Contacting System	1	Positive Pressure Injectors (PPI) supplied by Grace/Union Carbide	A total of 8 PPI heads each with a nominal capacity of 1 mgd, installed in a concrete tank having two compartments, each containing four PPI's. Each compartment has inside dimensions: 7'-6"W × 38'-1"L × 13'-10" deep with a water depth of approx. 10'
Ozone Destruct Unit	2	Heat-catalyst unit supplied by Mining Safety Appliances Company	Two units each designed to handle up to 248 scfm of gas containing up to 3000 ppm ozone
Control System	—	Supplied by Union Carbide	Ozone Gas Flow Control, Ozone Production Control, Makeup Oxygen Control, Regulation of Contactors
Indicating & Recording Instruments	—	Most of them supplied as part of the ozonation system by Union Carbide	There are several indicating and recording instruments but of significant importance are: three Dasibi Ozone Gas Analyzers, one Delta Scientific Dissolved Ozone Analyzer, and one LEL Monitor

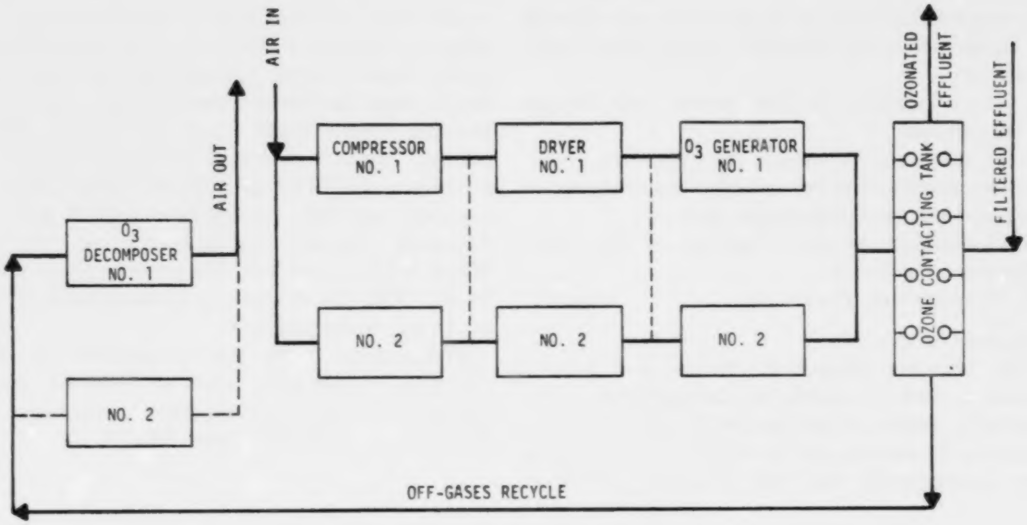

Figure 2. Open-loop Ozonation System Using Air as Feed Gas

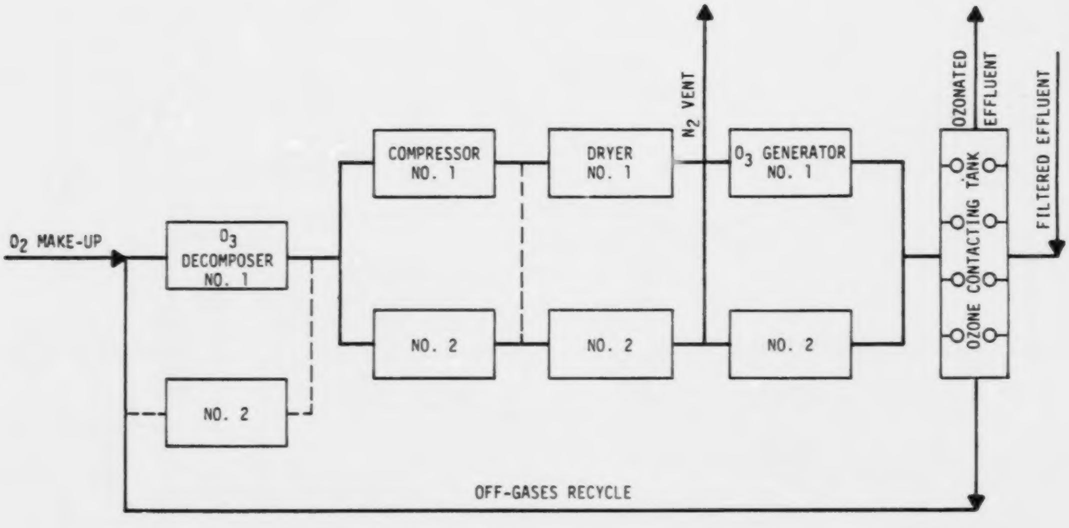

Figure 3. Closed-loop Ozonation System Using Oxygen Enriched Feed Gas

equipment. After the collection of background information, the efforts are directed towards achieving the objectives of the Meander Ozone R&D Study which are:

1. Determination of the power and oxygen consumption.
2. Evaluation of the off-gas recycle system.
3. Determination of the optimum ozone dosage to achieve a desired disinfection level.
4. Evaluation of the efficiency of the ozone decomposing device.
5. Development of cost data.

Instrumentation

The Meander Ozonation System was initially furnished with a variety of instrumentation for automatic control of the process as well as for data recording. In addition, several other instruments have been incorporated into the ozonation system to accomplish the objectives of the Ozone R&D Study. A process and instrumentation diagram (P&ID) is presented in Figure 4 which shows the total ozonation system which is being operated during Ozone R&D Study. Those instruments specifically provided for the Meander Ozone R&D Study are marked with an asterisk. Major instruments purchased are: one gas chromatograph (HP Model 5840A); one Dasibi ozone analyzer; one BIF venturi tube with Hagen flow indicating recorder; one sample pump (processor Model 9-1311); and one dew point analyzer (Alnor Model 7000L) for monitoring the moisture in the feed gas to the ozone generator.

Additionally, there are a number of other instruments/equipment such as pressure gauges, thermometers, and watthour meters, which have also been purchased for the Ozone R&D Study.

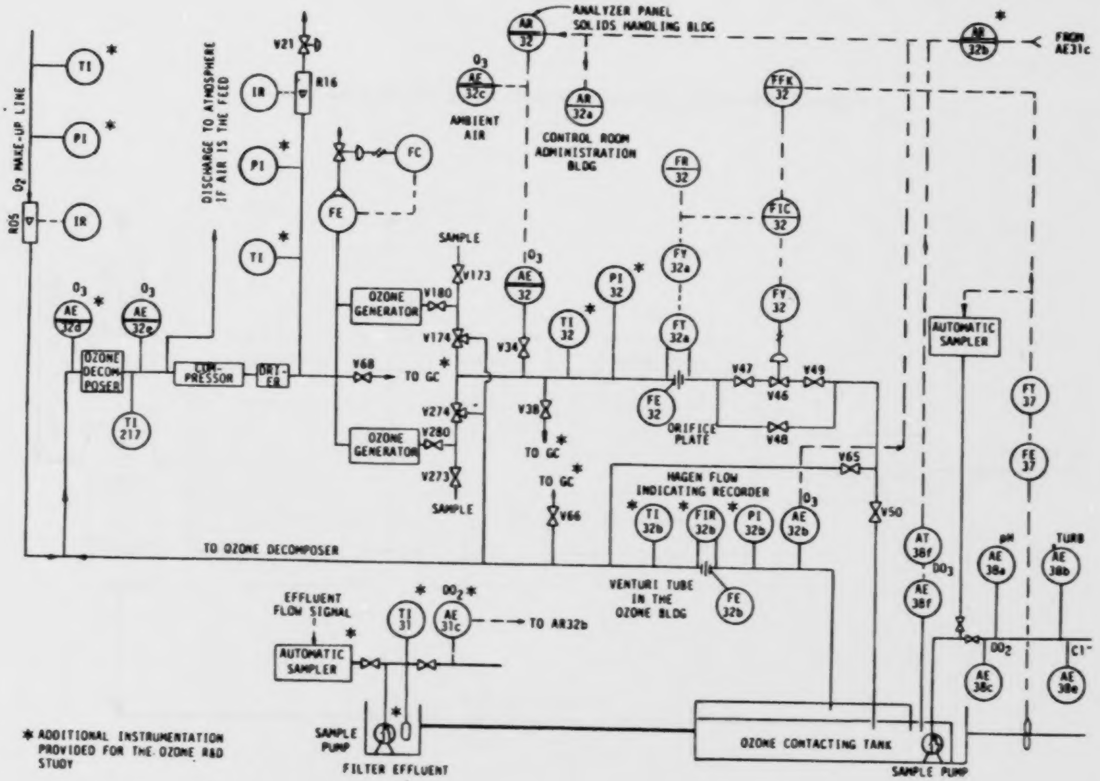

Figure 4. Process and Instrumentation Diagram for Meander Ozone R&D Study

PROBLEMS FACED IN CONDUCTING R&D STUDY

The ozonation system was originally supplied by W.R. Grace Company, who bid on a performance specification. Prior to completion of the installation, the ozone division was purchased from W.R. Grace by the Union Carbide Company. After a review of the system by its engineers, Union Carbide insisted upon making several major modifications to the system (at its expense) before they would assume full responsibility. These modifications resulted in delaying startup by almost two years.

Once startup was initiated, the usual number of other problems were encountered with each component. However, there were several major problem areas which resulted in extended delays in getting the system operational. The main problem areas of the ozonation system are: 1) Drying Equipment; 2) Contacting Tank; 3) Certain Controls and Instruments; and 4) Materials of Construction.

Drying Equipment

The drying equipment in the Meander Ozonation System is a refrigerative/desiccant type and is manufactured by Deltech Engineering Company. The incoming compressed gas passes through a porous stone prefilter to remove the finely divided water mist. The prefiltered gas is first chilled to about 7°C (45°F), at which temperature most of the contained water vapor condenses. The cooled gas is then passed through one of the two desiccant towers containing activated alumina held at about 7°C (45°F). The gas leaving the desiccant bed is about 7°C (45°F) with a minimum dew point of -40°C. Each bed remains in service for two hours. About 15 percent of this dry gas is heated to 52°C (125°F) and is fed back into the other desiccant bed in a reverse direction at atmospheric pressure. This action regenerates the out-of-service bed over a two-hour period by purging it of water it had absorbed during its previous cycle. Every two hours the towers alternate their functions and the process continues uninterrupted. The dried gas then passes through an after-filter. This is a sintered alumina with 5μm openings which is supposed to trap particles greater than 40μm in size.

Besides some relatively minor problems, three major problems in the drying equipment have been the refrigeration system, the drying media and the after-filter. The ozone generation equipment was installed almost two years ago but the drying equipment always gave problems. These problems were associated with leaks in the refrigeration system, moisture in the drying bed, and insufficiency of drying media. All of these components were serviced and replaced when necessary, but even so the desired dew point of -40°C has not been achieved on a sustained basis. Another problem area is the after-filter. The after-filter clogs often, necessitating frequent cleaning. Old after-filter elements have been replaced with new ones, but there is a feeling that the present after-filter openings (5μ) and length (41 cm)(16-inch) are too small and it would require coarser and longer after-filter elements to cut down the frequency of cleaning.

Ozone Contacting System

The ozone contacting system is comprised of eight Positive Pressure Injectors each with a nominal capacity of 3.8×10^3 m^3/d (1 mgd). They are installed in a concrete tank having two compartments, each containing four PPI's. Each compartment is 2.3m (7 feet 6-inch) wide, 11.6m (38-feet 1-inch) long and 4.3m (13-feet 10-inch) deep (inside dimensions). The water depth in the tank is normally about 3m (10-feet). The flow sequence in the contacting tank for the liquid and the ozone bearing gas is as given below.

The 76 cm (30-inch) concrete pipe carrying filter effluent to the ozone contacting tank is divided into two 46cm (18-inch) cast iron pipes at the south end of the contacting tank. Each of these pipes is reduced to 41cm (16-inch) and then further divided into four 25cm (10-inch) which are finally reduced to 15cm (6-inch) just before entering PPI. Ozone-bearing gas from the ozone generators is piped through a 10cm (4-inch) PVDC pipe to a 20cm (8-inch) central header which is connected to each individual PPI with a 5cm (2-inch) pipe. In each PPI, the filter effluent and the ozone bearing gas are mixed concurrently to create a selected gas to liquid (G/L) ratio which can be varied to optimize performance. This two-phase mixture is driven down a 25cm (10-inch) PVC pipe and the. released about 3m (10-feet) below water surface. The mixture then rises to the top of a 0.9m (36-inch) RCP concentric riser column where the gas and liquid separate. The liquid spills out of the columns onto a platform and drains away, passing under a baffle and over a weir before discharge. The baffle provides a gas seal to contain the ozone bearing gas. The gas is collected and piped back through a 20cm (8-inch) PVC pipe to the ozone feed gas preparation equipment for reuse in venting. Plan and section views of the ozone contacting tank are shown in Figure 5. Figure 6 presents a schematic of the PPI functioning.

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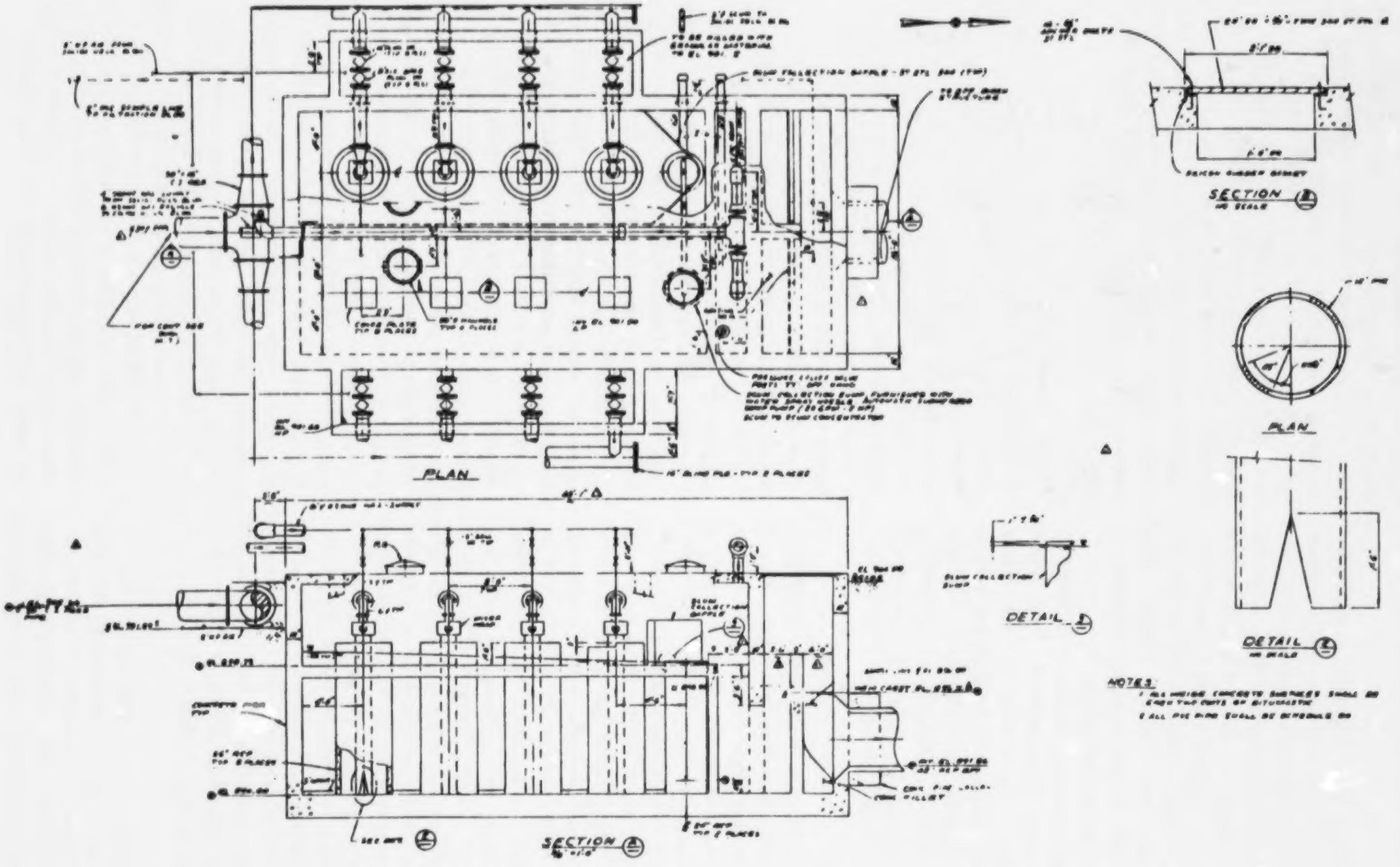

Figure 5. Plan and Sections of Ozone Contact Tank Meander Ozonation System

With the arrangement of the PPI contactor as shown in Figure 6, there was a possibility that during low filter effluent flow, or no flow condition, ozone might travel upstream and back up to the filter building where there was a potential for the equipment and the personnel to be exposed to the ozone atmosphere. Low flow and no flow conditions can exist as the effluent of the filter is used to backwash the filter cells. To alleviate this possibility, J-traps were installed in each PPI contactor. A schematic of the J-trap installed is shown in Figure 7. Under this arrangement, under low flow conditions, the ozone gas will have to overcome higher pressure though the J-trap (20cm) (8-inch), which will always be filled with water, as compared to the pressure to be overcome in the 25cm (10-inch) PVC pipe. The installation of the J-trap has completely eliminated the potential of ozone leaks in the filter building.

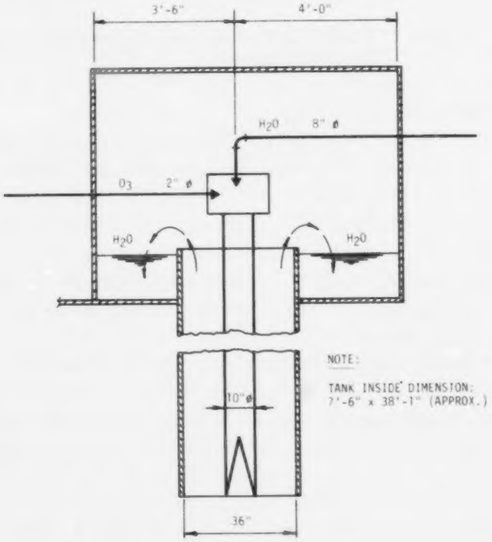

Figure 6. Positive Pressure Contactor (Original) Meander Ozonation System

Another problem encountered in the contacting tank is leaks in the PVC pipe. The 20cm (8-inch) central header, carrying the ozone bearing gas, is connected to each individual PPI through a 5cm (2-inch) pipe. The joints of these individual PPI connections with the central header frequently leak and consequently have to be frequently repaired. Initially, repair was done using plastic rod and compressed hot air to apply molten plastic around the joints. Later, plastic cement was used. Neither of the two techniques has proved completely successful. Perhaps a permanent solution resides in providing tee connections at these joints.

Another problem encountered at the ozone contacting tank is the occasional opening of the pressure/vacuum relief valves installed at the contacting tank. There is a 10cm (4-inch) pressure-vacuum relief valve at the contacting tank designed for 1m (39-inch) water column (WC) positive pressure and 76cm (30-inch) WC negative pressure. Although the cause has not been fully identified, it seems that when the gas to liquid ratio exceeds 0.25, the pressure relief valve opens. On the other hand, if there is a greater demand for oxygen at the make-up point than is being supplied, the vacuum relief valve opens, contaminating the system. These problems would indicate that the ozonation system at Meander can only be operated within a very limited range of parameters and cannot be subjected to a wide range of conditions without upsetting the operation of the system.

The ozone contacting tank has a total of four manholes and eight hatches for the removal of PPI heads. Gaskets provided with the original installation started leaking and had to be temporarily replaced with BUNA-N rubber. These were finally replaced with viton gaskets, which have proved extremely reliable.

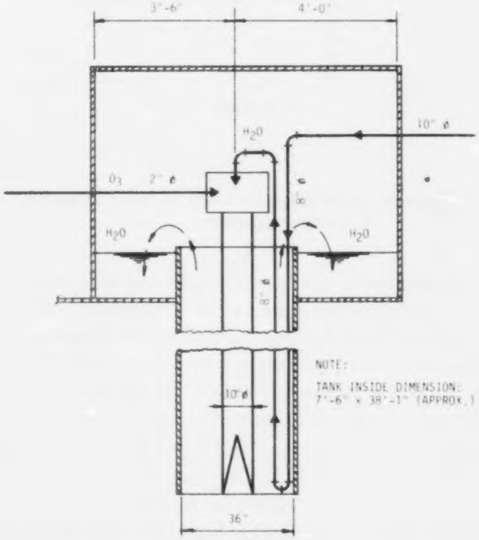

Figure 7. Positive Pressure Contactor (Modified) Meander Ozonation System

Controls and Instruments

Oxygen Recycle System

A schematic of the basic oxygen recycle loop is shown in Figure 8. There are three main sources of oxygen loss in the system which must be replenished in order to maintain a constant concentration of oxygen in the feed gas to the ozone generator: 1) Oxygen consumed in producing ozone; 2) Oxygen lost as dissolved oxygen in the final effluent; and 3) Oxygen lost in the nitrogen purge line. From the limited operating experience of the Meander Ozonation System, it appears that sufficient oxygen cannot be supplied for all operating conditions. Also, it is difficult to measure the amount of gas purged through the nitrogen vent line for all operating conditions.

Ozone Gas Flow and Production Control

Control of the amount of gas and the production of ozone is necessary to apply a predetermined ozone dosage to disinfect the final effluent. In the MOS, a signal from the final effluent flow transmitter can be used to control the amount of gas, concentration of ozone in the gas, amount of nitrogen purge, and the number of the PPI contactors in operation. Thus, the fluctuating demand of the varying plant flow is met by regulating these four parameters, thereby maintaining a predetermined dosage of ozone and predetermined concentration of oxygen in the feed gas to the ozone generators; and to open required number of PPI contactors for an optimum treatment. Such a control schematic is depicted in Figure 9. In this scheme, valve 46 (Figure 9) which controls the gas flow to the ozone contactor caused several problems. Initially, the valve supplied was too big for the system. It took several months for Union Carbide to realize that in fact this was the problem. An almost equal amount of time was taken to replace this valve by a smaller one. Even after replacement, the valve did not function well. Although the cause of malfunctioning of V46 could not be determined, it was made operational after a long period of trial and error. In the same control loop, the rotameter supplied to measure the gas flow through the nitrogen vent controlled by V21 (Figure 9) does not seem to be of adequate capacity. This poses a problem in that there is no measurement of the amount of vent gases when the capacity of the rotameter is exceeded and, thus, a mass balance of the gases in and out of the contactor cannot be made.

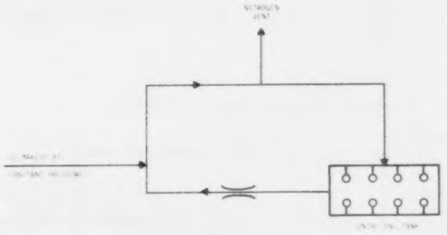

Figure 8. Basic Oxygen Recycle Loop Meander Ozonation System

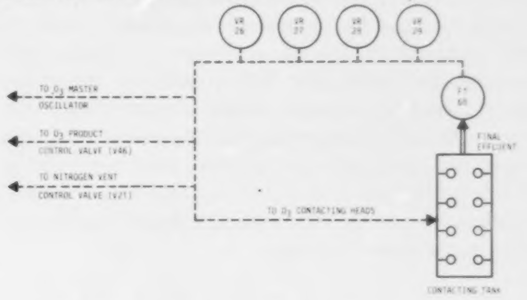
VR 26
VR 27
VR 28
VR 29
FT 68
TO O_3 MASTER OSCILLATOR
TO O_3 PRODUCT CONTROL VALVE (V46)
TO NITROGEN VENT CONTROL VALVE (V21)
FINAL EFFLUENT
TO O_3 CONTACTING HEADS
CONTACTING TANK

Figure 9. Control System Schematic Meander Ozonation System

Dasibi Ozone Meters:

The Meander Ozonation System has three Dasibi ozone meters. This instrument measures gaseous ozone by means of ultraviolet (UV) light absorption by the ozone molecule. Two of them are for measuring high ozone concentration in the exit gases from the ozone generator and in the off-gases from the ozone contacting tank. The third meter is for monitoring the ambient ozone concentration inside the ozone building. It appears that the electronics of these meters is good and reliable but the optics is not. The cell in the meter often fogs up with a yellowish film giving false reading of ozone concentration in the process gas. This film is very difficult to remove. Alcohol is not effective in this respect; however, we have successfully been able to fully remove it with sulfuric acid. The frequency with which these cells have to be cleaned is such that it defeats the purpose of spending money on the purchase of these expensive monitoring devices. Although unlikely, there may be some special contaminant in the gases at Meander which is causing this problem.

Delta Scientific Dissolved Ozone Meter

A dissolved ozone meter supplied by Delta Scientific is used at the MOS to monitor the dissolved ozone in the final effluent. This meter utilizes an amperometric membrane electrode for aqueous ozone determination. The Delta Scientific Ozone meter was delivered almost a year ago and installed in the final effluent channel. Since the system could not be made operational until recently, there was no opportunity to test the performance of the meter. However, when the ozonated effluent became available, the meter did not perform well. The meter has been sent for repairs. It is difficult to say at this time whether the problem lies in the design of the meter or its performance was adversely affected due to its long storage.

LEL Monitor

Although other safety features exist, monitoring of the hydrocarbons based on their lower explosion limit (LEL) is provided in the design of MOS to prevent buildup of hydrocarbons in the contacting tank and in the closed recycle loop. Should the hydrocarbon level reach a predetermined set point, the monitoring system provided will alarm, discontinue introduction of gas into PPI's, and turn on a purge air system. This monitoring system has been functional off and on for different reasons. Presently, the LEL monitor is not operational. The cause has not been determined. Union Carbide, supplier of the ozonation system, is working on it.

MATERIALS OF CONSTRUCTION

Initially, PVC pipe was used for both oxygen and ozone application in the Meander Ozonation System. In the present design, the use of PVC pipe is restricted. PVC can become brittle with age, and use of PVC in oxygen application is potentially hazardous in that the PVC ignition point is about 430° to 480°C (800° to 900°F). Although there are some exceptions, the use of construction materials in the MOS has been according to the following guidelines:

1. Copper or stainless steel is used in the application of wet oxygen.
2. PVC or stainless steel is used in the application of ozone and oxygen.

Use of PVC for ozone and oxygen is restricted to underground applications. However, some exceptions had to be made. For economic considerations, the main header and the individual pipes feeding ozone to PPI's are made of PVC and are above ground. However, in these pipes, leaks at the joints have been a constant problem.

Initially, teflon tape was used as a lubricant/sealer in the joints of stainless steel and steel pipes. However, it was found to be inadequate for sealing the joints. Teflon tape with RTV cement gave an effective seal and it has been working satisfactorily for the last year without any problems. The original gaskets (material not known) in manholes and hatches were found very unsatisfactory; they could not stop leaks in the tank. They were replaced by gaskets of BUNA-N rubber for temporary use. Now the gaskets used are of viton (fluoroelastomer) and have been performing satisfactorily for the last year.

Although hypalon (chlorosulfonated polyethylene) is recommended for ozone applications, the hypalon diaphragm in an ozone meter pump was reported to have been attacked by the ozone. The manufacturer considered replacing it with a viton diaphragm.

DATA OBTAINED

As previously discussed, the ozonation system at Meander AWTF has been operational only a short time and, therefore, only one week's data are available for reporting here. Data pertaining to ozone disinfection efficiency are given in Table 3. Although it is difficult to draw a definite conclusion from such little data, it appears that ozone dosage of 4 mg/l should be sufficient to achieve a disinfection level below 200 fecal coliform/100 ml. This dosage is 67 percent of the design dosage of 6 mg/l. All the disinfection data were obtained using a gas to liquid ratio in the range of 0.18 to 0.27. The plant flow during this study varied from 6400 to 8300 m^3/d (1.7 to 2.2 mgd).

Data on the quality of the influent to and effluent from the ozonation tank for the period of the disinfection study are presented in Table 4. As can be seen from the table, the influent to the contacting tank was of very high quality; BOD ranged between 0.6 and 1.8 mg/l, COD ranged between 5.1 and 28.6 mg/l, and TSS ranged between 1.2 and 3.4, except in one case where the TSS concentration was 12.8 mg/l. There is an indication of a slight increase in BOD of the ozonated effluent. Concentrations of COD in the ozonated effluent remained mostly unchanged except in the last two cases where there was an abrupt increase. These results cannot be explained and this increase can only be attributed to analytical errors. In the case of TSS, there was an increase in five out of seven cases. This might explain in part the reason for increased BOD in the ozonated effluent. There was an increase in dissolved oxygen in the ozonated effluent, which was to be expected, as the ozone was produced with gas feed containing oxygen in the range of 45.3 to 61.2 percent. Ozonation, at least for the ozone dosage applied in this study, does not seem to affect the pH of the effluent as can be seen from the data presented in Table 4.

Data pertaining to power and oxygen consumption are presented in Table 5. These data do not show any trend. However, average oxygen consumption was 8.85 kg per kg of ozone produced and power consumption averaged 18.1 KWH per kg (8.24 KWH per pound) of ozone produced. These values correspond to an average oxygen concentration of 51.94 percent in the feed gas to the ozone generator. They appear to be high and more data will be required to develop reliable values.

CONCLUSIONS

Based upon a short operating experience with the Meander Ozonation System and from analysis of the very limited data available, the following conclusions might be suggested:

1. The Meander Ozonation System supplied by Union Carbide (previously W.R. Grace), even after making extensive modifications and changes, has not performed reliably to date.

TABLE 3. OZONE DISINFECTION EFFICIENCY, MEANDER OZONE R&D STUDY

Sample Date August 1978	Flow mgd	Ozone dosage mg/l	G/L (a)	Influent Total Coliform per 100 ml	Influent Fecal Coliform per 100 ml	Effluent Total Coliform per 100 ml	Effluent Fecal Coliform per 100 ml
23	1.7	3.38	0.27	480,000	20,000	3,400	260
24	1.7	3.12	0.24	700,000	220,000	3,700	600
25	1.8	3.13	0.24	800,000	80,000	10	6
28	2.2	4.15	0.19	140,000	4,600	760	8
29	2.1	4.79	0.20	45,000	3,000	150	25
30	2.0	6.57	0.21	54,000	2,300	490	20
31	2.0	8.38	0.18	>1,000	<1,000	1,000	15

(a) Gas to Liquid Ratio.

TABLE 4. WASTEWATER QUALITY, MEANDER OZONE R&D STUDY

Sample Date August 1978	BOD mg/l		COD mg/l		TSS mg/l		pH units		NH_3 mg/l		DO_2 mg/l	
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
23	0.6	3.0	28.6	28.6	2.4	5.4	7.08	7.4	0.6	0.6	8.0	11.4
24	1.4	3.0	28.3	18.9	3.4	5.4	7.59	7.70	0.6	0.7	7.1	14.0
25	1.1	1.3	56.6	56.7	1.2	3.4	7.44	7.40	7.3	6.5	7.1	11.8
28	1.7	2.9	5.1	5.1	12.8	18.2	8.24	8.04	<0.1	0.1	7.7	11.2
29	1.2	1.3	20.2	20.2	1.4	1.6	7.88	7.99	<0.1	0.1	8.2	11.3
30	1.8	2.4	20.4	91.8	2.8	1.6	7.88	8.10	<0.1	0.1	8.2	13.5
31	0.9	—	20.0	60.0	2.6	0.8	10.19	10.16	0.3	0.4	7.8	10.2

TABLE 5. OZONE GENERATING PARAMETERS, MEANDER OZONE R&D STUDY

Sample Date August 1978	% Oxygen Feed Gas	Dew Point Feed Gas, °C	% Ozone Produced	kg Oxygen/kg Ozone	KWH/kg (lb) Ozone
23	50.4	—	0.87	—	25.5 (11.6)
24	54.9	—	1.01	12.1	22.4 (10.2)
25	52.0	—	1.22	7.6	11.9 (5.4)
28	47.2	0	1.84	4.7	17.0 (7.7)
29	45.3	-59	1.97	14.0	16.1 (7.3)
30	52.6	-59	2.59	9.0	11.9 (5.4)
31	61.2	-50	3.35	5.7	22.1 (10.0)
Average	51.9			8.85	18.1 (8.2)

2. Except for some surface discoloration, no visible material degradation has been noticed so far in the PVC pipe used in the Meander Ozonation System. However, use of PVC in ozone applications should be carefully investigated for each individual case. Stainless steel, although expensive, would be a more durable and reliable material in such applications.
3. Viton (fluoroelastomer) is highly recommended for gaskets and diaphragms in ozone applications. Hypalon (chlorosulfonated polyethylene) appears amenable to ozone attack.
4. Operation of an ozonation system with recycled oxygen feed is a fairly complex system for the disinfection of treated wastewater effluents.
5. Average consumption of oxygen and power per kg of ozone produced on 52 percent oxygen feed are 19.47 kg (8.85 lb) and 18.13 KWH, respectively. These values are almost twice the values normally expected from such systems.
6. Initial indications are that disinfection to meet the 200 fecal coliform/100 ml discharge can be met with an ozone dosage of 4 mg/l.
7. In its present condition, the oxygen concentration in the recycle feed gases to the ozone generator cannot be achieved more than 60 percent.

22.

ROUNDTABLE DISCUSSION OF OZONE DISINFECTION

Participants in Ozone Roundtable

1. Mark C. Meckes
   U.S. EPA-Cincinnati
2. Albert D. Venosa
   U.S. EPA-Cincinnati
3. Edward J. Opatken
   U.S. EPA-Cincinnati
4. K. Rakness
   M&I Consulting Engineers
5. Rip Rice
   Jacobs Engineering
6. Jain S. Jain
   Engineering Science, Inc.

MR. VENOSA: Rip Rice has asked that he be given a couple minutes to make a few points. So, we will start off with Rip.

DR. RICE: Thank you, Al. From the discussions we were hearing yesterday afternoon, I thought it would be smart to make some general points which we found in our survey of drinking water treatment plants using ozonation systems and using ClO_2 systems.

Most of the comments that I am going to make are applicable directly to ozonation but some of them are also applicable to ClO_2 systems, too.

There are something like 1,050 drinking water treatment plants throughout the world using ozonation systems and these have been in constant use since 1906. One of these plants in Nice operated actually 60 years using the same ozonation hardware prior to upgrading it.

More than 500 of these plants are in France and most of the rest are in Europe. The USA has only five, Canada has 19 or 20. The important point is that in sewage treatment with ozonation the U.S. is pioneering this application. The Europeans know all about how to generate ozone, dry air, contact it for drinking water, but they know very little about its use in sewage treatment. So when you hear the horror stories, keep in mind that we are on the cutting edge here and, number two, that Union Carbide is pioneering a brand new type of ozonator and a brand new type of contactor.

I have a list of some of the plants in the USA that are involved with ozone (slide). There are six or seven of them which are operational. That means they are either operating or they are in the startup phases.

The Indiantown, Florida plant has been operating this November for three years, constantly, without having to maintain either the ozonator or the contacting system, and this is the longest operational sewage disinfection plant using ozone in the world. It is a very small plant. The asterisk over at Chino Basin means that this is the one plant in this group that is not using ozone for disinfection. It is using it for suspended solids removal.

The next overhead shows those plants which are in construction using ozone for disinfection. All the other plants other than the one asterisked are using ozone for disinfection, and here we have six plants, all of which are being built, about to be started up. The next slide shows some of the plants which are under design using ozone for disinfection. There are 16 here. Six of them have been named. It is not right to name the other 10 yet because there is, as you know, competition among the manufacturers.

The asterisk against the Cleveland Westerly Plant means that they are not using ozone for disinfection here but for pretreatment of water prior to passage through dual media filters, one medium of which is activated carbon which is now biologically active.

One of the things that I think is important is that in the European drinking water treatment plants the normal dosage of ozone for any purpose averages between 1 and 4 mg/l. During the survey which we made we did cost estimates of several of the larger European drinking water treatment plants and we found that the cost for ozonation systems, cost for air preparation, for generation, for contacting, for instrumentation, for control, for the analyses, for the operation, the maintenance, all factors, amortization and so forth, costs ran between 1.75¢ per thousand gallons to 4¢ per thousand gallons.

We found the major variable factors in the cost for ozonation systems were twofold. These are variables now. The first one was the housing that you put the ozonation system in. If you go out and build a brand new plant and build a separate housing for the ozonation system as opposed to taking an existing plant and jamming the ozonation system into it, that gave us a broad range in the cents per thousand gallons.

The other one is the cost of power which over there varies from 2 to 5¢ per kilowatt-hour.

Another point that I think is worth making is that in drinking water treatment practice multiple uses of ozonation are being made. In other words, at the end of the process if they are using ozone for disinfection or bio-inactivation, many times they will take the off-gases and put them in the front end of the process with the chemical addition to aid in bringing down suspended solids. This gives them a much clearer, less turbid solution to take onto the plant.

In terms of sewage treatment disinfection, it is my understanding that the recommendation to this point is that secondary effluents should be filtered prior to ozonation. In that case, I would think that it would be wise to look at taking contactor off-gases and not throwing them up the stack but putting them up in front of the filter step. This should do several things. One is to help you knock down suspended solids. Also, partially oxidize some of that COD into BOD and if you can adjust the flow rates through the sand filter you will have some biological degradation and nitrification. This should give you a better quality effluent if not a somewhat more constant effluent, but that is argumentative and I am not saying that there is any data to show that to be the case.

Finally, in Berlin, Germany I think there is a new approach being taken in sewage treatment. The pilot plant is taking primary sewage effluent, ozonizing it with 1 to 2 ppm dosage of ozone, not for disinfection but merely to partially oxidize BOD and COD, make it more biodegradable, saturate the sewage with oxygen, and then that effluent goes into the ground, into a sand filtration system in the ground and then on out into the rivers. So, effectively what they are trying to do is "secondary treatment" in the sand prior to the water going out into the river. That is kind of interesting and so I wanted to make those general comments.

MR. VENOSA: Thank you, Rip. Do we have any comments from the audience now? Okay, Harvey.

DR. ROSEN: Well, first I would like to apologize to the other manufacturers when I said that you should have called me for information. I meant you should have called us. We are not the only manufacturers. There are many of them here and we all provide a lot of information and do a lot of hard work to provide information.

With respect to the particular economic analysis done by Ed Opatken. I called the home office and I got authorization to sell what I understand to be the $280,000 system under Table 7 . . . that is, equipment cost for ozone generation, for compression, and drying . . . . for $65,000 to anybody in the room. That is the current market price. It is one unit. It is not 28 units. It is skidded. It goes in with a forklift truck. It is about 5 feet by 12 feet, takes almost no room as far as the building space involved; I do not know what to say beyond that.

Rather than picking on any individual parts of the analysis, as Ed indicated when you do this analysis, that capital cost for the ozone equipment is the overriding factor which comes down to affecting the bottom line, and what that does, if you change nothing else except that capital cost, (and, by the way, we will warranty better operating costs than are shown in this analysis) comes down to is: look at the capital investment under diffuser 15. We chose this because that was the point of 90% utilization. That is another thing which manufacturers will warranty, at least we will if we are involved in a design.

Ozone utilization is not a black art. It is mass transfer. It is gases dissolved in liquids and things like that and we think we understand it enough to put up or shut up in terms of penalties and warranties with respect to that, as well as all the performance that you would expect with respect to how many pounds a system will make, how much power that system draws, and those sorts of things. We even warranty reliability up to a point more than just the one year normal manufacturer's warranty of parts and labor, because they tend to be new things and people are concerned about the problems with new equipment.

To corroborate that number, I do not know if the gentleman from the Washington Suburban Sanitary Commission was involved in our recent negotiations to sell him a system that is similar to what I am talking about. It is something that comes off the shelf. It is called an MA-75. With a device to decompose ozone and with diffusers and some other things on a rental basis, I think the price would be $82,000, if after it was all rented out over a year or something like that the Commission decided to keep the equipment.

So, these are the kind of prices there are available on the marketplace. There are, of course, as Dr. Rice showed, a lot of plants being bid and that is where to go out and find out how much these things cost. There is competition out there, and you will find on a dollar per pound of ozone basis that you get a pretty good price now compared to even a couple of years ago, because there is some volume in sales and manufacturers can afford to reduce the cost of manufacture and sell systems at a cheaper price.

The particular system that I am quoting, the MA-75, is a 75 rather than a 62 pound a day system. They come pretty much standard. So, it is even a little more ozone that we are talking about that was required for this particular application here.

With respect to the general technology development and innovative technology, I do not think, as Rip Rice mentioned, that it is fair to judge what the future costs will be necessarily on the first one or two start-ups. We have a plant that was listed up there at 35 mgd at Springfield, Missouri, which has started up, and had minor start-up problems relative to most wastewater treatment plants or any plant that size which includes over 3,000 pounds a day of ozone. I do not want this to get into a commercial, but I do want to put things in perspective. That is the third plant now as opposed to the other two that we have been involved in, and the performance characteristics are embarrassing relative to the warranties because of the improvements made between the time we sold the equipment and the time it was installed and working. These are some of the things I was trying to say yesterday.

There is over 94% utilization of the ozone and these are warranties or acceptance tests done by Birdsall and Jenkins, and all the standard methods that are done by the consulting engineer when he accepts the equipment, and tests that system.

So, I think we have to take this whole thing, regardless about whether it is ozone or UV or any of the new technologies and be a little careful about . . . and this was mentioned by one of the UV gentlemen yesterday . . . . before we start to condemn them too early. Thank you.

MR. BOB FLEISCHER: I did not have to call the home office. I did some evaluation in my own head last night and my evaluation is based on actual bids. In fact, the numbers I am going to use are actually higher than actual bids that took place less than six months ago.

For instance, there were two 23 kilogram units with several metering devices that I would say retailed for approximately $4,000 apiece. I think there were three on this job, and two ozonators. In other words, redundancy . . . . went for $69,000 on the marketplace.

DR. RICE: 23 kilograms per hour or per day?

MR. FLEISCHER: 23 kilograms per day, excuse me. And this job is an odor control job on Long Island. I can discuss it with you further if you would like. It was not my company that got the job. It was another manufacturer, as a matter of fact.

However, based on Table 6, I went back just figured on what capital costs would be, power would be, etc., for that size plant. I was extremely conservative. For instance, I have enough money in here to build a house with a refrigerator and a freezer and stove around my ozone generator, and total cost with full redundancy is $104,000.

MR. OPATKEN: Installed.

MR. FLEISCHER: Installed, piping . . . .

MR. OPATKEN: Housed and everything . . . .

MR. FLEISCHER: Housed, diffusers, contactor, labor, everything. . . .which at first cut. . . .and this is just the first cut in my head, brought the cost down from 361 to 209, which is a 42% reduction, and I did not even breathe hard.

MR. OPATKEN: Good. You also instrumented it with Dasibi's and so forth?

MR. FLEISCHER: That is correct. There are over $10,000 worth of meters that I threw in. I would like to comment that I do not really want for anybody to put the blame anyplace. Nobody ever called me and I am the man who receives these type of phone calls at PCI. There is no way that I would ever recommend anybody scaling up from anybody else's ozone generator, including my own. On top of that this job looks like, from the regeneration numbers, oxygen feed, which is absolutely absurd for 65 pounds a day of ozone. That is not real world.

We have several plants that are going on for as little as 3 pounds a day ozone disinfection and in which the engineers are finding it cost effective. There are two projects in West Chester County and one is running, it is called Hunter Highlands, I will take you up there, 3 pounds a day ozone for a condominium complex. Another one is Oak Ridge which is in West Chester County. Saratoga Waste Water Treatment Plant is using ozone at 37 pounds a day. That is even smaller, yet the engineer has found it cost effective.

I could go on and on, but I would rather not. I would like to make one other comment and this goes back to the previous study.

We at PCI feel that the type of diffuser used for the, I think it was 85% mass transfer, really is not the best diffuser available today and that higher mass transfer values can be gotten with a cylindrical type diffuser. If that was used, mass transfer would be better and perhaps even the amount of ozone utilized would be greater versus the amount of ozone applied.

MR. OPATKEN: Where is your data on your diffuser analysis?

MR. FLEISCHER: If you look at Bollyky's mass transfer paper. . . .

MR. OPATKEN: All right, I talked to Dr. Bollyky at the last EPA meeting.

MR. FLEISCHER: Well if you look deeply into his paper, you will find it.

DR. JOHNSON: Isn't this a kind of an unending argument? That is, naturally, as the technology improves and if you compare government specs, with all due respect to the government well represented at the front, we all recognize that when you build something according to government specs, I mean it is a different type of deal than if you do it yourself.

So, cost is cost and I think this is an unending argument. I would like to go back to a more fundamental question.

Let me first preface this by stating, I think that ozone is probably the best method for killing viruses and other very difficult to disinfect microorganisms such as cysts, of all of the various methods of chemical disinfection available. At the same time, ozone is very reactive. It is a very strong oxidant. We all recognize that. It is rather selective in its reaction with organic compounds, primarily reacting with unsaturated compounds as we will hear more about later today, I think, from Dr. Jolley and Dr. Cumming.

But the basic problem with ozone is control, in my estimation. We have a very strong oxidant, that is very reactive, and we are putting it into sewage, which has a lot of compounds in it that the ozone can react with, which will take and pull the ozone off to reactions other than what we would like it to do, which is kill microorganisms.

The side reactions may actually consume all of the ozone that we have put in. In fact, in many cases it is very likely to consume all the ozone. Even if we had done as the manufacturers would like for us to do, (run pilot plant studies, determine on site with the actual waste what kind of demands we have for ozone and what kind of dosages we have to put in with that particular contactor and that particular waste and that site), who is to prevent manufacturer A from dumping a little phenol that particular day into the waste stream and suddenly we do not have any disinfection going because all of the ozone we added in our pilot studies without the phenol coming down the pipe is suddenly being consumed by what we got that morning. That is a real problem for any kind of wastewater disinfection facility.

This is the question then. How do we control ozonation processes so that we can be sure that, on a continuing basis where we are controlling with dosage alone, we have a continuing disinfection process going on? It is a very difficult question for ozone, and I think the question that we need to focus on in controlling the ozone disinfection process on a continuing basis so that we have assurance on a continuing basis that we have disinfection and not some side reaction occurring.

MR. OPATKEN: I would like to make one statement. During my paper I said that what I used was the pilot plant unit and the production required was based upon that pilot plant unit being a modular unit and scaled up in terms of units. I obtained those units through the mapping curve and I said earlier that I was not concerned with the optimization of ozone disinfection costs. I was concerned with ozone contactor evaluation and economic evaluation. The differential costs could be adjusted accordingly if you have an ozone generator that can produce this at a lower kilowatt-hour per kilogram. That is an advantage and you can take it right into these costs and apply it. If you can get more production out of it, fine. You can then adjust your fixed capital investment, but for me to be sitting there at the pilot plant, I have to estimate a fixed capital investment. I did it on the best possible method I thought fair. That is the data that I have.

Now, as I said when I started my paper, I think the consultants are smart enough to be able to specify, and we have tried to help them. Al gave a paper saying that so much of this is necessary, at so much concentration to get this level of disinfection.

I have not heard the ozone manufacturers come out with that type of data. They always come out with the fact that we will give you something. Now, the consultants are also smart enough to take that mapping curve and specify where they want to be. They then have the knowledge to them to start up that plant.

One of the difficulties we have in ozone is that a lot of this information is with the manufacturer. Then we get to the field and this manufacturer has that information. The engineer in the field is blind. He doesn't know where he's supposed to be.

We were given an ozone system for our pilot plant without a dew point meter. We were ignorant at the time, too. Now, I would not recommend any facility without a dew point meter. We were given a dryer that supposedly had a fixed orifice. We could not dry our air. We had to readjust our regeneration. We had to add a rotameter. So we had to apply control methods.

Now, I have seen plants out in the field and they also were lacking in some of these fine operational details, and so when you say they are skid-mounted and so forth, that is all well and good, but remember that I laid everything on the line when I calculated out my number of units.

Now, these units are based upon that curve. I have asked one of my fellows to duplicate that curve so that each of you can have a copy of it. All I am saying is, let the consultants do the analyzing instead of the manufacturers hiding this information among themselves. I think you ought to have an open competition here.

DR. ROSEN: I think if you were at an ozone bid at any time you will find that there is a very stiff competition.

MR. OPATKEN: And whatever you are offering is hidden.

DR. ROSEN: No, it is not hidden as a matter of fact. It is very nice to offer that curve, but that curve is for your unit that you are using in that laboratory and that is part of the problem with putting out that information. It will become a gospel. It will become a standard for all ozone generators, and although you say no, and I am telling you that is what will happen and it is happening in the marketplace. I guess in somebody's file this is an ozone generator. Generically it does not make any difference what type is used. We design and provide information relative to a particular problem at a particular plant.

Now, in our brochures we will be publishing for our equipment on a relative basis, some of the kinds of curves you are talking about, because again we are talking about covering a large size range of equipment and all performing the same. It has to be on a relative basis as opposed to an absolute basis, but as the equipment gets harder and we learn more about it, we do publish that information in brochures. You know, it is not even that secret.

Another point is that if you change that price in that particular diffuser (15 milligrams per liter ozone concentration) and just make the adjustments on the basis of that capital price, you come down to 6.3¢ per thousand gallons rather than 13. So, when I said a factor of 2, I was not too far off.

MR. RAKNESS: I would like to speak for a minute on the gentleman's question about operation of a plant and what do you do to continue to get disinfection. At the Upper Thompson plant, since we are sure we need disinfection, what the operators do if there is any doubt at all, is to crank the ozone production up probably greater than what is optimum from a cost standpoint, and get disinfection levels much less than what are required. Now, that does not take into account any industrial dumping and things like that, but. . . .

DR. JOHNSON: What do you do about the big demand that comes down the pipe you do not know about?

MR. RAKNESS: Well, the Upper Thompson facility does not have a lot of manufacturing plants that are hooked on the industrial sources. It is a residential community with commercial tourist-related type wastewater. It does not have industries that would dump things like phenol.

DR. JOHNSON: Unfortunately, most of our facilities are not that well off.

MR. RAKNESS: Well, I just wanted to make the comment that we do probably dose at a higher level than what is needed to just achieve our disinfection level.

DR. JOHNSON: Of course one answer is to set your dosage up here when most of your needs are down here, but that is not very cost-effective, is it? You hope that your excursions of demand do not ever reach your dosage level.

DR. ROSEN: Oh, yes. I do want to try and answer your question. Part of the problem with what Ed was saying is that you can instrument and control and do all kinds of things with an ozone system or any system for that matter to any degree to which you are willing to pay for it, and there are a lot of sophisticated. . . .

DR. JOHNSON: Wait a minute. There are measurements to make. How are you going to instrument it and control. . . .

DR. ROSEN: You know how. You are developing a technique yourself. There are a lot of other people. . .

DR. JOHNSON: That is why I asked the question.

DR. ROSEN: I know, but there are a lot of other people also developing techniques. One of the techniques that we are looking at and Al has told me he will be looking at is, if you are changing ozone demand, you are changing the amount of ozone coming out in the gas phase because more of it is being reacted or less of it is being reacted as compared to that which you are applying, and it is easy to measure ozone in the gas phase.

The question is, can we take that delta, how sensitive it is, and can we relate that back to demand and disinfection? That we do not yet know. Now, that method of control is used in European plants to control a constant dissolved amount and has been used successfully for a number of years.

DR. JOHNSON: Okay, but in that method of control where we are working with the difference between the ozone in the gas phase coming in and the ozone in the gas phase coming out, that only tells you how much ozone is going in the solution. It does not tell you whether it goes to the microorganism or whether it goes to the demand, and if it all goes to demand, then you are lost as far as the microorganism is concerned. So, what are you going to do?

DR. ROSEN: Well, isn't that true of chlorine or anything else?

DR. JOHNSON: No, with chlorine we measure residuals and we control on the basis of residuals. When the residual goes down, we up the dosage.

DR. ROSEN: Someone said the other day, that from a practical point of view, that is more a crutch than a practical solution.

MR. WHITE: That is general practice.

DR. ROSEN: I know it is standard practice, George. There have been a number of reports in the literature where people have studied that standard practice with the chlorine residual that the state recommended and Don, I think, will agree that they have not achieved the disinfection standard.

MR. WHITE: That is how we achieved disinfection standards in California and I wanted to say that nobody has defined disinfection. If 200 fecal coliforms per 100 milliliters is the standard, you could do better with a hammer. That is not disinfection.

Disinfection is a graded thing. There is a certain water requirement. There is a confined estuary requirement. There is a negative estuary requirement. There is fish breeding place requirement. There is water sports requirement. They are all different. They imply certain grades of water quality coming out of the treatment plant and the disinfection is to a certain number of total coliforms.

DR. JOHNSON: I am a native Californian and I would like to make one point and that is Californians are crazy.

MR. OPATKEN: We have a grant with the city of Marlborough to look at ozone for disinfecting at what we consider high levels of disinfection. So, that is down the road.

MR. WHITE: Right, but the thing is when we talk about disinfection, we have to define what we mean. That is all I am saying.

DR. ROSEN: As I heard someone once express it, and I do not think it is a bad way of thinking of it necessarily, wastewater treatment is the preparation of wastewater to be disinfected. So, the up-stream processes and the quality are important, regardless of disinfectant, because they all have other demands (UV, chlorine, ozone). If you do not have a relatively good water to disinfect, whatever your definition of disinfection is, it creates a big problem and that is part of the overall problem that I think we are discussing here.

DR. RICE: Don, I would like to add a little and maybe clarify. There are actually two control items here. One is the ozonation system itself which can be controlled very well to put out any amount of ozone in any concentration at any rate you want. What you have asked really is the major critical question, and this is one of the things I had in mind when I said we are pioneering the use of ozonation in sewage treatment.

In the ideal ultimate, you should never think of pacing disinfection by ozone dosage, ozone demand, because that implies you know what the demanding components are. In the ultimate end, in my opinion, the ideal way is to have a real quick bacteriological test, which we do not have.

DR. JOHNSON: Well, you cannot grow bacteria instantaneously.

DR. RICE: That is correct. Therefore, in my opinion the best approach is the last comment that Harvey made which is to make sure that the treatment process which you are going to use is good enough to handle the components which may come into that sewage in preparation for disinfection. That is a crutch and is not an ideal situation, but that is where we are with ozone.

DR. JOHNSON: I will direct this to the panel (and this is where I would like to see some more data from Mark and Al and the kind of studies they are doing now). I was encouraged to find that somebody had found some ozone residual in wastewater. The first time I had heard that anybody has been able to strip out ozone from a wastewater contactor and then measure some ozone after a gas phase transfer in another vessel was in what Mark found. We went up and talked about the data.

That is encouraging and I think that is where we are going to have to go. I am sorry that Al's comment was that you could not correlate the ozone residuals that he had found with his disinfection. That is too bad. Again relying on our ancient history from chlorine, we know that sometimes chlorine residuals do not correlate very well with disinfection and I think the reason for that is we are measuring something other than chlorine residuals. In other words, the analytical method is getting us in trouble, particularly when we measure total chlorine. We take a lot of iodide and we take the pH down and we measure a dichloramine fraction, which, I agree with George, is probably in large part not really inorganic ammonia-type chloramine.

Bob Baker did some nice work on the Philadelphia supply, looking at chloramine residuals and their various components. The dichloramine fraction really was not dichloramine. It was organic chloramines, which are probably not very good disinfectants.

So, I hope that we can start to do some measurements and I hope that you all will put into your data, even though it does not correlate very well, the different ways you measured ozone residual and how it did correlate if at all. That is where we are going to have to go with the ozone field if we are going to answer this question of what to do with demand as it goes up and down, other than hitting it with a huge amount of dosage which we cannot afford to do.

MR. VENOSA: I would like to say one thing about ozone residual.

The magnitude of the residual has a lot to do with the type of contactor being used. If you have a countercurrent flow configuration, you can have higher residuals. With the co-current flow configuration, residuals hurt you because the higher the residual the higher must be the off-gas concentration; thus, mass transfer is inhibited.

DR. JOHNSON: Mark, can you give us some of the data that you had on residuals and make a few comments that are appropriate to be made about how you measure residuals. Somebody back there talked about the Shuval-Schechter method for measuring residuals in which you add the KI directly to the wastewater and measure the iodine generated with spectrophotometry. There you are measuring all the oxidants in the solution and that is very different from what you find in the Standard Methods. . . .the book that nobody pays attention to but everybody refers to.

It says in Standard Methods that you must, before you measure ozone residual, strip the ozone out of the solution and into another container, and that the Schechter method does not do that.

MR. MECKES: Don, I cannot give you results off the top of my head. However, we did test residuals by amperometric forward titration methods at pH4 and amperometric back titration methods at pH4. We did do a method that was done several years ago in a couple of previous studies whereby it is a colorimetric type measurement, directly taking residual and then adding postassium iodide to produce the iodine, which then you titrate. We found this was very erratic.

The backward and forward titrations measured quite accurately in water. However, when we got to wastewater, we could correlate the two only sporadically. Sometimes they would be right on the money together and other times they would not. We did do a stripping method as is reported in Standard Methods. However, we put the unit on-line so that we had no waiting period from the time we drew our sample until the time we purged it with air to release any ozone to the gas phase into our potassium iodide washing bottle. These gave us extremely low results. However, we were able to get some measurements from these and at this point in time I really feel that there is still a lot of work that needs to be done.

We did not look at Schechter's method.

DR. JOHNSON: It really makes little difference how you measure the iodine generated. Whether you use Schechter's method, whether you use amperometric titrations or what you use. The problem is getting the ozone measurement separate from the oxidant measurements and stripping is one method for doing that.

Dr. Jain, did you get any data with ozone electrodes?

DR. J. JAIN: We bought it but it was not operational. We had to send it back for repairs.

DR. JOHNSON: You got the very first one, I think.

DR. JAIN: I guess so. I guess we got everything first.

DR. JOHNSON: That is the trouble with pioneering.

DR. JAIN: I would like to make a couple of comments before we divert the subject, not for the analysis but for the ozone generator as a whole.

I do not mean to knock down the salesmen, but they can sell you anything. Don't really believe them what they say. I am being honest. We have had the experience. I have salesmen friends who are fine gentlemen. They are really professional, but yet they are under pressure to sell you something. Also I have been in the business for the last six years and I have known ozone development in this country. All these people who are the key people from four or five firms, have been changing with each other like musical chairs.

Now, you go and talk to one person who asks. . . why did you come here and you say the reason I came was because the others were not right enough. The data they gave was not right, and I could not justify it.

Now, at this point Ed Opatken's data may sound a little on the high side but for the consulting engineers, for others who are really wanting to use this data, this is good data. You will find it is more useful than what the salesmen are telling you now.

For example, at Meander they state that intially we were supposed to have four ozonators for the total capacity. Only two ozonators could be accommodated. For the next two we had to have more building. At the time they told us that the ozone machine would be of this size and you can have four ozone machines without any difficulty.

Power costs. At this point the indication we have is that the kilowatt on the air costs us something like 18 kilowatt-hours per pound of ozone. In Mahoning we are talking something 8 to 10 kilowatt hour per pound of ozone. The capacity, rated capacity, the maximum we could get was something like 150 pound per machine as opposed to 237 pound, and this I have in writing. It is not just talking. It can very well be that it is the first machine, but still they guarantee.

Now, even though they guarantee you, you go to the specification. We have penalties. How do you really apply those penalties? Could you go into court? You are under pressure, you want to make the system work. Nobody's interested in going to court. So, it does not really help. Those penalties in the long run are so minor, for the project to start it takes almost 3 to 5 years and maybe $25,000. . . .they say they do not care for that. They made the money.

So, please be sure, before you take a number, you have to have confidence in yourself, not what they are telling. They are telling because from their interest they want to sell you anything.

An example is this: at Meander we have been struggling three years. We have people with W.R. Grace, who are the same people with Union Carbide and they are helpless to help us. For example, coming back to the Springfield plant. I was told the Springfield plant is working fine and I suppose it is working, but it is very different from Meander. They are not using recycled oxygen for producing ozone. Instead they have a once through system. They are treating their off-gases to the UNOX system. So this is different.

When we had a meeting with these people they told us the control system is the same. If the control system is the same, instrumentation the same, why doesn't Meander work? They have tried and it has not worked so far. We are still praying, we are praying from different mediums, and it has not worked.

MR. RAKNESS: I would add to that by saying that there is such a thing as innovative technology and I think that that is where we are now. Six years ago, or so we first looked at design, and two years ago we installed this and that. If you are thinking as design engineers and as communities that are going to put in ozone systems, you now are probably looking at some innovative technology.

What I would recommend to you is to understand the entire system and talk to different companies, different manufacturers, different people who had been involved with it, and understand the entire process and all the in's and out's about it. At that time, make your own decisions as to the approach to use and the things that you can rely upon. Some of the major things I consider are to look at two different conditions: the design condition and the initial operating condition. That is what the client sees when you first start up. As I showed on the graph yesterday, the initial power utilization was much greater than at design conditions by a factor of half again as much. That is what the consumer initially sees when you start up the system.

So, understand the system, talk to different people, and make your own decisions as to what approach to take.

DR. RICE: I have one word . . . amen.

MR. FLEISCHER: I have several comments and I want to make them very quickly. The first is that, as I said before, nobody came to me. I have had power curves available for you for two years that are free and open to the public.

MR. OPATKEN: It is not necessary to come to me. Go to the consultants. . . .go to the industry itself.

MR. FLEISCHER: The consultants have these curves when they ask for them and with all due respect to Dr. Jain, I have been dealing with Dr. Jain on Cleveland Westerly for two years. My power numbers have not changed for two years. My capital costs have not changed for two years, and in fact, I was told that a building for Cleveland has to be built with a second floor that PCI will not even utilize because the other manufacturers need it for space.

I agree that the manufacturers can vary in power requirements because their generators are different. Harvey represents Union Carbide. I represent PCI. These are two state-of-the-art ozone generators that do not nearly consume as much power, but Dr. Jain is stuck with the situation, in a bid situation where he may not have a Union Carbide generator or PCI generator. He may have one that consumes more power, and there is a problem. How do you penalize somebody who already has a generator on-site?

The second comment I would like to make is that for at least the two years that I have been with PCI, dew point indicators or analyzers are standard on our equipment, and, in fact, I have not seen a specification come from a consultant that has not required, whether it be standard or optional, a dew point analyzer. It is just as plain and simple as that. We have never put out one without one.

MR. OPATKEN: Did you have one on yours, Kerwin?

MR. RAKNESS: No.

MR. OPATKEN: There is an answer.

MR. FLEISCHER: What size generator was it?

MR. RAKNESS: 76 pounds per day.

MR. FLEISCHER: That is not true. There is a dew point indicator.

MR. RAKNESS: There is a color changing indicator that suggests as the dew point reaches the point where you have moisture. . . .

MR. FLEISCHER: That is the dew point indicator which is standard on smaller models; on larger models when you get over 100 pounds a day, then you get into analyzers.

MR. RAKNESS: That is a good point, but what I want to add is that when the operator looks at the dew point indicator and sees a color change, then he knows something is wrong. He can look for a problem.

The problem with the color changing indicator was that it was not sensitive to gradual changes in dew point and you could not sense a problem as quickly as you could with the dew point analyzer. The problem with the dew point analyzer was that it still required an operator to look up and observe a change in dew point and that was checked once per day, every morning, and between the morning check and the next morning check, the generator could flood and did flood.

That is where we got into the idea in the recommendation that we made in the talk yesterday that consideration be given to a high dew point alarm or an automatic shut-off that prevents the generator from flooding.

MR. FLEISCHER: I agree with you. The indicator is just as you said, but this is typically what is required in the smaller type of units. When we are getting to the point of 100 pounds or more ozone, then the dew point analyzers are typically supplied instead of the indicator.

By the way, the consultant is always made aware of what is available. He is told specifically that that is an indicator and what it does and that if he wants an analyzer, it will cost him in the 100 pounds a day probably less than 1% of capital cost to add an analyzer and that is his option.

MR. OPATKEN: Yes and I think that is great because this technology that we are transferring today is telling this consultant to make sure he does have a dew point meter that is not a color changer.

DR. RICE: I have a point to make on dew point meters. In the European ozonation systems they say there are two reasons for having a dew point monitor, not an indicator.

First is you want to make sure you have low dew point to make the most ozone at the lowest power cost. The second is, which to me is even more important, is to lower the maintenance requirements. Once the air goes above -40°C, then you are going to have to maintain the ozone generator, because you are going to have moisture getting in there and I am not sure exactly what all happens. What they say is, as long as we are monitored at −60°C and lower, we are only going to have to maintain that ozone generator once a year if that. Thus, once the indicator says you have exceeded the dew point, it is really too late in terms of maintenance. The damage has been done.

DR. HILL: I just wanted to compliment the EPA's work on the contactors. The comparison of the three types of contactors was something that was needed for some time. Regarding the talk about the family of curves on ozone production, I would like to make a comment on that, showing one slide.

What this slide shows is the performance of a laboratory ozone generator, the Welsbach 223, as a function of pressure. People who have discussed ozonator performance so far have mentioned that power, as it is controlled by voltage and frequency, is very important and they have also talked about how the gas flow is very important.

In this experiment the ozonator gas flow was maintained at 0.1 standard cubic foot per minute of oxygen and the primary voltage, that is before it is transformed to high voltage, was maintained at 100. The only thing that was changed was the pressure within the ozone discharge reactor, where the ozone is generated. In the far left you can see that at 1 psig the concentration of the ozone is 37.5 mg/l, and then when you come upt to 8 psig, it is down to 20 mg/l. So, for this type of ozone generator there is a very big difference in pressure. It amounts to 85% increase in ozone generator production in going from 8 psig to 1 psig. I believe this particular manufacturer rates its equipment at 8 psig. This is an important consideration.

Now, not all ozone generators work better at a low pressure. Each ozone generator has one optimum pressure, and in the more traditional designs that have a long tube and about 3 inch diameter they usually have a discharge gap of about 3 millimeters. This discharge gap is one of the most important things in determining what the optimum pressure is.

For these types of traditional generators they seem to work a little better at low pressure. Some of the newer designs such as the PCI and the Lowther plate that Union Carbide sells, have a smaller discharge gap. They are made to operate at their optimum pressure at a higher pressure, closer to what you would need to go into a tall column of water.

I just wanted to make this caution. Those people who are using these families of curves should make sure that the pressure is the same when they attempt to use that in their equipment. A good reference I would like to recommend, if someone is interested in finding out how an ozone generator works, is a dissertation by a fellow named Anthony Edelman. It came out in 1967 at a university in the Netherlands, and where I found it was at the Library of Congress, but it may be available other places.

MR. WHITE: As a special advisor to consultants, we have a problem and I think we are missing the point here. When a consultant specifies a piece of equipment, he is going to expect the manufacturer to perform, and I have no reason to believe that any of the ozone manufacturers, in spite of what we heard today, would not want to perform. Why keep yammering about disinfection limits when we start talking about disinfection. I am sure we are not going to agree. The people in California are not going to agree with the people in Alabama, or for instance, Vancouver, British Columbia, where they think 10,000 per 100 milliliter coliforms is disinfecting their sewage. So, we can just forget all about that and all these requirements in disinfection.

I am working on two projects where I am recommending a combination of ozone and chlorine and what I want to know is: what is the ozone consumption on a well oxidized domestic sewage? I need to know this and we just do not have any performance data that we can hang our hat on. We have to go back to performance data that is on potable water which is relatively clean and that is all we have. So, what we need is more do and less talk.

We need this performance data and I do not know how we are going to get it except having installations where we can actually observe the operation. I might add that one of the ways that the people I talked to in France operate their ozone equipment is that they routinely make ozone demands everyday and they have a historical record of the ozone demand of the water that they are treating. This goes back to the same thing that we have done in sewage chlorination. We get a historical record. We know what the chlorine demand is, except that we do have the advantage of being able to control from residual and this hampers the ozone situation. But I need information, and that is the kind of information I need. I need cost per pound of ozone and somebody has to tell me how many pounds of ozone per day I need to put into an effluent to achieve a certain disinfection.

MR. VENOSA: I will give you a copy of my paper, George. It is all there.

MR. WHITE: I have a copy of your paper.

DR. LONGLEY: I want to give credit to Archie Hill. He is coming up with what appears will be some very interesting information. His report should be written in about two months. He is under contract to the U.S. Army Medical Bio-Engineering Research and Development Laboratories. So it is not a proprietor-oriented report. Hopefully, it is a report written with great objectivity.

I want to develop a little on what Don was saying a while ago, and I want to speak as a member of the Standard Methods Committee. Don happens to be the chairman of the Joint Task Force for Chlorine Residual Techniques.

I think what we have discussed here a little bit in the past few days is some of the problems with residual analysis. At least it keeps rearing its ugly head. As I understand, one of your objectives of this symposium might be to identify future research needs. I suggest that what we need, whether it be an inhouse EPA effort or whether it be an extramural effort, is to look at the analysis of oxidants and to start developing some of the information that we need on accuracy, precision, the problems that we have with the various techniques, their applicability, and we can talk forever about it. I think this is an important area that has been overlooked, at least in the recent past insofar as the research dollar and the research effort is concerned.

MR. OPATKEN: We are working on a grant agreement between Miami University and EPA in regard to the residual analysis. That is underway at this time.

DR. RICE: On this subject, extending it a little further and back into drinking water where the example is, the French use ozone for many reasons, one of which is virus inactivation. What they do is attain an ozone residual level of 0.4 mg/l, and after attaining that, maintain it in a plant for 6-10 minutes. Then that guarantees polio virus 1, 2, and 3 have been inactivated. This is a performance disinfection standard which was first adopted in Paris and is now adopted throughout France. France is the only country which has formally adopted that.

The other countries that are using ozone for disinfection of drinking water are doing the same thing. The extrapolation is that if one would take the time to pretreat sewage to the point where you can get residual ozone and would have confidence that there are no hills, valleys, particles for the viruses to hide, one could then use that technique for virus inactivation.

I am not recommending that in sewage treatment because I think that would cost a fortune.

DR. JOHNSON: That is something chlorine cannot do. So, I think that we want to put a little encouragement into folks particularly after they have been discouraged as Dr. Jain has by all of his problems, because ozone has some real potential. It is an excellent disinfectant.

MR. NOVAK: Rip, I would just like to add to what you are saying. I agree with you that there is monitoring equipment that is being used now and, just to add to what Harvey said before, they do measure ozone concentrations out of the ozonator going into the system. They measure the off-gas, but they also can measure residual ozone in the water. As Rip just said, they measure 0.4 ppm ozone in the water after 5 minutes and consider that a disinfected water.

DR. RICE: A virally inactivated water, which is also bacterially disinfected, but they do not have a bacterial disinfection standard. It is only that viral thing.

MR. NOVAK: The work that Dr. Rook has done also indicates that the bacteria kills are no longer there.

DR. RICE: Well, yes, but how many fecal coliforms do you have in the drinking water?

MR. NOVAK: If you look at raw water supply and if you are taking water from the Rhine you are talking about a sewage treatment plant. They do phys-chem. They do promoted sedimentation. They have laminar flow devices, sand filtration, activated carbon. They treat it just like a sewage treatment plant. So there is not that much difference. Mr. White, you asked for data. We have published data on that Indiantown plant that has been running for about three years now, which gives a chemical analysis, biological analysis. It gives residual ozone dose, residual ozone absorbed in the entire system. So you have some back-up data for engineering design.

One other thing on contactor systems, when we talk about killing bacteria, we first built our plant with a bubble tower type reaction chamber and we ran into major problems with that type of an injector system. It would run very well for approximately 1-2 weeks, 3 weeks, and then all of a sudden we would start to get some types of slime growths on the side of the wall and we could not get consistent, reliable bacteriological kills. We would have to shut the whole system down, scrape the walls, and then start our system up again, and we would get good bacteria readings for approximately 2-3-4 weeks.

I admit it was in a tropical climate in Florida. It got very hot and very humid and that may not be the case all over the United States. But we switched to a very, very rapid contacting system and were able to get consistent bacteriological kill with one minute contact time of ozone and the water and the wastewater with very low dosages. We reduced the ozone dose rate from about 12 to 14 parts per million down to about 4.5 to 5 parts per million. If we use a single stage reactor we can get transfer ratios of about 80 to 90%. If you go to a two-stage reactor you can get 100% utilization of the ozone.

MR. DeSTEFANO: I have a question for Mr. Venosa that is sort of related to this residual question. Is ozone a surface active oxidant? What do we know about the action of ozone versus chlorine on cell surfaces or, is it surface or is it in through the membrane or what?

MR. VENOSA: I have absolutely no idea.

DR. RICE: If you have a surface which can be oxidized, yes, ozone is surface active, and so are all of the other oxidants as well.

MR. DeSTEFANO: Right. I am sure a lot of the engineers are familiar with the fact, maybe more through water treatment than with surface charges and surface-surface interactions, when you are doing coagulation control and maybe adding alum and a polymer, you are trying to neutralize the negative charge on whatever is in there. They are all colloids.

One of the jobs that I have with the firm I am with is to keep up research on this because we manufacture an electrophoresis instrument. I will not mention the name, I am not trying to sell them. It seems to me with any surface-active oxidant that you have to get past whatever is absorbed onto the cell before you are going to kill the cell, and I am not sure if any research has been done, maybe not in this field but maybe in microbiology, about how much choice the cell has on what absorbs onto its membrane on a macro scale, talking about big proteins that are present in wastewater. I have seen research done with albumin on clays and what you get with albumin absorbing onto a clay is a high buffering capacity, where say a dosage 1,000 parts per million of salt would salt out or coagulate a clay. If you absorb a certain amount, maybe 1,000 ppm of albumin or just put egg white in it, you can go up to maybe 20,000 without coagulating.

DR. RICE: I am not aware of any research of this sort in sewage treatment where they are using ozone, but there is some in the drinking water area, notably pesticides in the presence of added Kaolin or suspended solids, something that can adsorb the pesticide on the surface. Now, without the adsorbing substrate, the ozone takes the pesticide which I think was Aldrin which is very reactive with ozone, down to zero in a very short time, but adsorbing it onto the substrate makes it much more oxidation resistant. It took more ozone, took more time and so forth.

MR. DeSTEFANO: But what I am more concerned with is the smaller molecules adsorbing onto the larger cells. Now, Dr. Johnson was saying that you cannot get complete correlation between residuals and disinfection. He thought that that is probably because you are not measuring the residual correctly. Well assume you are measuring the residual correctly, it seems to me that possibly the protein, or unless you have something that is chlorine resistant and is going to buffer the cell against the reaction of the hypochlorous acid or hypochlorite ion or whatever you consider the disinfecting ion, that is going to be the limitation to disinfection rather than a strict chlorine residual.

It is something that I admit is hard to measure and I am not even sure you would be able to use this as a process control. Maybe as consulting engineers keep in mind when you are talking about disinfection, you try to imagine yourself as an *E. coli* floating through the plant and what you see and what is happening to you when you go through. . . .

MR. VENOSA: Thank-you. We will keep it under advisement!

DR. LONGLEY: What the gentleman I think is talking about is mechanisms and we probably need some more mechanism work. Vincent Olivieri, who was here yesterday, was talking about ClO_2, has done mechanism work on phage. It is interesting work. It is long and hard and tedious and what comes out of it is ultimately useful. Well, I think it is, but apparently EPA does not because they do not want to fund that type of research right now.

QUESTION: I have a question in connection with analytical procedures. I was wondering whether anybody has had any experience utilizing methodology which FMC has published using a titanium sulphate solution acidified with sulphuric, and then diluted. It is a colorimetric reaction which works very well for unreacted hydrogen peroxide. If it works on that, it should work pretty well on ozone too, since basically the chemistry is the same. Has anybody had any experience with this methodology? The reason I am asking is I am the new chairman of ASTM D-22 for ozone and we do not have a method.

I am guessing here but the problem is not that you cannot analyze for strong oxidants. The problem is that as the strong oxidants react with substrate materials, you get a lot of weaker oxidants produced and it is very difficult to distinguish among this spectrum of oxidants. The problem is to analyze for a strong oxidant like ozone and the presence may include peroxide and God knows what other organic peroxy-compounds of a variety of types of varying reactivities.

MR. WOOD: I would like to make a comment here. I am somewhat amazed by a reaction that is described and not giving the end product. We hear people talking about putting controls on sensing flooding of a reactor with the answer that you shut that unit down in case of that malfunction. We have to put equipment in and one of the stipulations is in no way can we shut down that equipment. In watching a sewer plant operation, you cannot shut off disinfection. So, sensing it and shutting down the piece of equipment is not the end of the reaction because you just cannot in my opinion do this.

You have to cut in the standby piece of equipment which means you do have to have a standby.

MR. RAKNESS: That is right and that is what we have. So it is shutting down one and turning the other one on.

DR. JAIN: I want to make a couple of very brief comments. I want to make myself clear that I am for ozone. Ozone has a big future. In fact, Rip Rice, Harvey Rosen, and I started at the same time in the business.

The reason why I am complaining about Meander and keep on harping on that is because it is hurting the cause of ozone. It is in the interest of everybody that Meander should work. If it does not work and we have to go to chlorine, it is going to hurt ozone and not any particular company like Union Carbide.

The second thing was regarding the ozone analyzer. We have ozone analyzers now especially for the gas phase, Dasibi. This is a very expensive piece, we are talking $4,000-5,000. In a plant you need several of them and you also would like to have a standby. So, this is one area where we need research and development of instruments which are cheaper and perhaps more reliable. The Dasibi ozone analyzer does need maintenance.

The last one, before anybody makes recommendations particularly regarding the materials we use with ozone, they should have some experience. For the last six years we have been hearing PVC is good for ozone, unplasticized. You use it. You write it in specification. Your problems are solved. This is not true. Estes Park has the experience, we have the experience. Please do not specify unplasticized PVC for ozone use.

DR. RICE: On that last point, as a general rule of thumb, from our survey, the people in Europe say dry ozone in dry air can be handled in anything. Wet ozone in wet air should be handled only in a very costly stainless steel environment but in the long run you do not have those maintenance problems.

SECTION 6. VIRUSES AND ORGANICS

23.

VIRUS INACTIVATION IN WASTEWATER EFFLUENTS BY CHLORINE, OZONE, AND ULTRAVIOLET LIGHT

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ABSTRACT

Five wastewater treatment plants were compared in their ability to inactivate naturally occurring enteroviruses. Two of these five plants used chlorine, two used ozone, and one used ultraviolet light in the disinfectant stage of wastewater treatment. Correlation coefficients between measured virus concentrations and total coliform, fecal coliform, turbidity, TSS, COD and TOC are presented along with a study of the diurnal variation of virus emissions in both winter and summer at one plant. Polio, Coxsackie A, and Coxsackie B viruses have been identified and their relative concentrations are presented for all five wastewater treatment plants. Virus isolation rates and virus concentrations were shown to be significantly reduced by all three modes of disinfection.

INTRODUCTION

This report is a summary of findings for a field and laboratory program conducted by The Carborundum Company to determine the virucidal effects of chlorine, ozone, and ultraviolet light under actual operating conditions at five wastewater treatment plants. The work is sponsored by the U.S. Environmental Protection Agency (EPA) and greatly aided by the inputs of Mr. Albert D. Venosa, EPA Project Officer, his associates and staff, as well as the on-site personnel with whom we have worked at each Wastewater Treatment Plant. Mr. Robert Fluegge is Principal Investigator. Dr. Theodore Metcalf, University of New Hampshire (UNH), Dr. Craig Wallis, Baylor University, and Dr. Joseph Melnick, Baylor University, are consultants to this program. Carborundum Field Engineers responsible for all field work include Messrs. Richard Thacker, Ben Moore, Kevin McConnaghy, and Charles McGee.

The field portion of the program required extended sampling at four wastewater treatment plants. These were located in New Jersey, Colorado, Ohio, and Massachusetts. The laboratory assays were completed at facilities in New Hampshire. Program coordination was located in Niagara Falls, New York.

WASTEWATER TREATMENT PLANTS

Four wastewater treatment plants were studied during this program. A total of five separate sample collection periods were completed between November, 1976, and May, 1978.

Waldwick (Northwest Bergen County) Operations

The Northwest Bergen County Wastewater Treatment Plant, Waldwick, New Jersey was sampled during the spring of 1978. It used a conventional and/or step aeration activated sludge system followed by disinfection with ultraviolet radiation (for this study) with the final effluent treated with chlorine. A block diagram for this plant is shown in Figure 1.

This plant had a design flow of 3.2x10^4m^3/d (8.5 mgd). However, the total plant flow during these studies was only 1.9x10^4m^3/d (5.0 mgd). More details of this plant are given by O. Scheible (2).

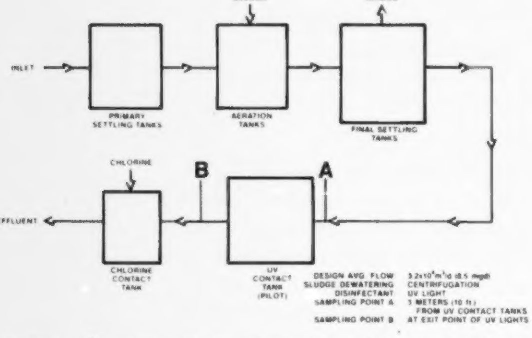


FIGURE 1. NORTHWEST BERGEN COUNTY SEWAGE TREATMENT PLANT: N.J.

The ultraviolet light source was a prototype unit manufactured by Pure Water Systems, Inc., Fairfield, New Jersey. It housed 400 100-watt lamps with 30% power at the germicidal wavelength of 254nm. Each lamp was contained in quartz jackets measuring 1.5m (60 inches) in length bay 23mm (0.9 inches) in outside diameter. The quartz jackets were spaced 12.7mm (0.5 inches) apart at the surfaces. The contact time was 3.6 seconds at a flow rate of 1.9x10^4m^3/d (5.0 mgd).

After passing through the ultraviolet disinfection stage, the liquid effluent was treated with chlorine as required by the State of New Jersey before it was discharged into Ho-Ho-Kus Brook.

Estes Park Wastewater Treatment Plant Operations

The Estes Park Wastewater Treatment Plant, Estes Park, Colorado, consisted of a conventional activated sludge plant followed by nitrification, tri-media filtration, and disinfection by ozone (air was used for ozone production). The design flow was 5,700 m^3/d (1.5 mgd), but the actual flow was 2800 m^3/d (0.75 mgd). It was sampled during the spring of 1977. A block diagram for this plant is shown in Figure 2. A detailed description of the plant is given by Rakness and Hegg (1).

Ozone was produced on-site from air using a Welsbach corona discharge generator. The contact tank was a rectangular diffuser chamber divided into nine equal sized compartments by baffles. Wastewater effluent flowed over and under the baffles in serpentine fashion, providing cocurrent and countercurrent gas-liquid contacting. The contact time was 37 minutes during the virus testing period. The average dosage applied during the period was 4 mg/l.

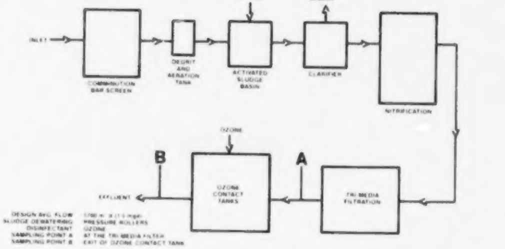


FIGURE 2. ESTES PARK SEWAGE TREATMENT PLANT: COLORADO

Muddy Creek Sewage Works and R. A. Taft Laboratory Operations

The Muddy Creek Sewage Works, Cincinnati, Ohio was a secondary wastewater treatment facility using a conventional activated sludge system followed by disinfection with chlorine. Separate tests were performed using ozone as a disinfectant in December, 1976, and chlorine in July, 1977. A block diagram of this plant is shown in Figure 3.

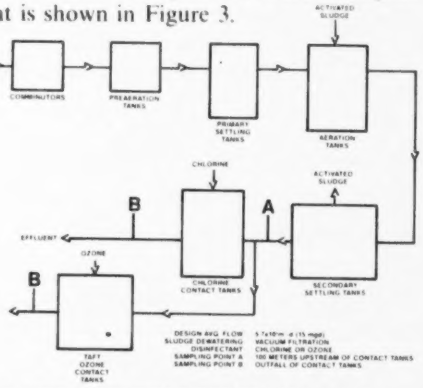

ACTIVATED SLUDGE
INLET
COMMINUTORS
PREAERATION TANKS
PRIMARY SETTLING TANKS
AERATION TANKS
CHLORINE
ACTIVATED SLUDGE
B
A
EFFLUENT
OZONE
B
CHLORINE CONTACT TANKS
SECONDARY SETTLING TANKS
TAFT OZONE CONTACT TANKS
DESIGN AVG. FLOW 5.7x10⁴m³/d (15 mgd)
SLUDGE DEWATERING VACUUM FILTRATION
DISINFECTANT CHLORINE OR OZONE
SAMPLING POINT A 100 METERS UPSTREAM OF CONTACT TANKS
SAMPLING POINT B OUTFALL OF CONTACT TANKS

FIGURE 3. MUDDY CREEK SEWAGE WORKS: OHIO

Ozone was incorporated as an alternative disinfectant for part of this study. Unchlorinated secondary effluent was transported in a 20 m^3 (5,400 gallon), epoxy-lined tank truck to EPA's R. A. Taft Laboratory pilot plant. The effluent was pumped into the laboratory building and through a packed column ozone contact tank. The applied ozone dose during the test period was 8 mg/l. The average residual was 0.13 mg/l. Contact time was 30 seconds at a liquid flow rate of

75 l/min (20 gpm). A full description of this facility is given by Venosa *et al* (3).

Marlborough Easterly Wastewater Plant Operations

The Marlborough Easterly Wastewater Treatment Plant, Marlborough, Massachusetts was a conventional two-stage activated sludge system with nitrification followed by disinfection with chlorine. Effluent samples from this plant were studied in October, 1977. A block diagram of this plant is shown in Figure 4.

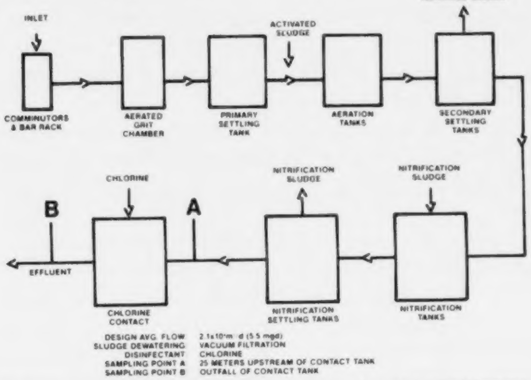


FIGURE 4. MARLBORO EASTERLY WASTEWATER PLANT: MASSACHUSETTS

The chlorine contact tank provided a contact time of 50 minutes. The chlorine residual averaged 1.3 mg/l during the test period.

A summary of effluent quality characteristics for each of the five study sites is presented in Table 1.

Sample Collection and Assay

A Carborundum mobile laboratory was driven to and remained at each site during the extended sampling periods. Figure 5 is a side view of this laboratory on-site at Estes Park, Colorado. The unit was fully self-contained with onboard generators, water tanks, air conditioning, sinks, and storage facilities for chemicals and equipment.

FIGURE 5. CARBORUNDUM MOBILE LABORATORY ON-SITE AT ESTES PARK, COLORADO

All sample collection and treatment was accomplished inside this mobile laboratory. The surrounding environment was controlled to eliminate spurious

TABLE 1. SUMMARY OF EFFLUENT QUALITY CHARACTERISTICS AT THE FIVE STUDY SITES

Parameter	Bergen	Estes	Muddy Creek	Marlborough	Muddy Creek/Taft
TSS, mg/l	5.3	8.5	2.7	12.6	7.4
COD, mg/l	30.2	36.1	27.9	24.4	39.2
TOC, mg/l	13.0	—	11.6	6.5	11.2
Turbidity, NTU	2.1	5.0	1.8	5.0	4.0
Disinfectant Dose	131,000 (μ-watt-sec/cm^2)	4.2 mg/l	—	—	7.9 mg/l
Disinfectant Residual, mg/l	—	—	1.2	1.3	0.13
Contact Time, minutes	0.06	37	51	47	0.5
Total Coliforms per 100 ml before disinfection	1.8×10^5	8.4×10^5	1.65×10^5	1.8×10^4	7.8×10^5
Total Coliforms per 100 ml after disinfection	3.7×10^1	3.8×10^3	6.5×10^3	2.3×10^2	3.1×10^3
Fecal Coliforms per 100 ml before disinfection	4.1×10^4	6.9×10^5	7.8×10^4	1.8×10^4	1.9×10^5
Fecal Coliforms per 100 ml after disinfection	2.4×10^1	3.3×10^3	1.0×10^3	8.4×10^0	1.2×10^3

inputs from the nearby waters, atmosphere, and general on-lookers. As will be evident later, the virus concentrations were quite low and hence required care in sample taking.

Carborundum's Aquella Virus Concentrators were mounted inside this laboratory and used throughout the program.

Sample Collection and Virus Concentrations

The Aquella Virus Concentrator was a portable, self-contained apparatus designed to remove waterborne virus from large sample volumes on a continuous basis. The purpose during this phase of the experiments was to field process approximately 380 liters (100 gallons) of wastewater per sample and to concentrate the sample to as small a volume of liquid as practicable for analysis to facilitate handling and shipping problems.

All sample hoses, pumps, filter holders, and tubes were rinsed, sterilized, and rinsed again prior to collecting each sample. Dilute hydrochloric acid (HCl) at a pH of 0.5 was used to sterilize all components. The HCl contacted the components for 30 minutes prior to final rinsing. The sterilization procedures have proven to be very successful, especially when sampling potable waters in the vicinity of highly contaminated waters such as typically found in raw wastewater areas.

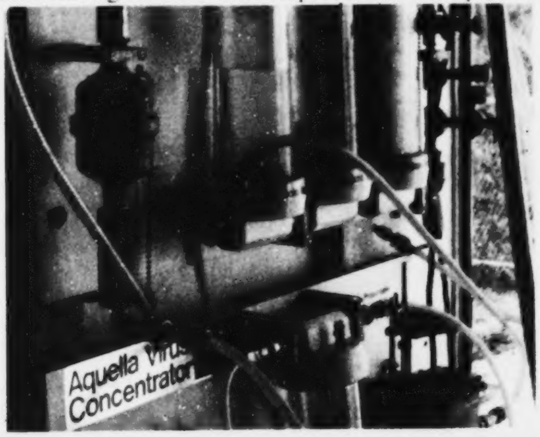


FIGURE 6. WALL MOUNTED AQUELLA™ VIRUS CONCENTRATOR UNIT

Two units are mounted on the side walls of the Carborundum mobile laboratory used for this program. Figure 6 shows one of these units bolted in place on the passenger side wall of the van shown in the previous figure. This arrangement is somewhat different from the standard configuration in which the free-standing unit is connected together with hinges and end panels for ease of handling and shipping. Since this van was used at all wastewater plants studied, the units were permanently mounted.

The laboratory walls, ceiling and floor were disinfected prior to collecting each sample. Chlorinated water was sprayed on all surfaces and allowed to dry. In addition to laboratory sterilization procedures, Carborundum Field Engineers were trained to insure personal cleanliness. This is especially important while sampling sewers, settling tanks, and aeration chambers. Every effort was made to eliminate cross contamination between the equipment, the laboratory and the on-site personnel.

Wastewater was collected by placing a submersible pump in the center of the flow stream being sampled. Water was pumped through one or several nylon garden hoses to the inlet of the Aquella Virus Concentrator and entered at the right of Figure 6. A flow rate of 3.8 l min (1.0 gpm) was typical in this experiment. Water continued through an impeller pump and was further forced through three clarifier filter holders which may or may not have contained filters. The requirement for filters was primarily governed by solids in the wastewater. Several times during these experiments, clarifier filters were required due to particulate loadings. Once through the clarifiers, water flowed through a chemical proportioning pump which had several satellite liquid injectors. Conditioning chemicals were added at predetermined rates which were automatically controlled by volume flow. This feature became important as filters plugged and flow rates correspondingly decreased.

Sodium thiosulfate, dilute HCl and $AlCl_3$ were added through the satellite liquid injector chemical proportioning pump. The exact concentrations were dependent upon the wastewater characteristics, but our researchers and consultants have shown good virus recoveries for an adjusted pH between 3 and 4, **$AlCl_3$ concentrations near 1.5 mM, and sufficient sodium thiosulfate to eliminate active chlorine, ozone, or other oxidizing compounds.**

A flow totalizer was mounted in the flow stream just prior to the two custom designed virus adsorbing filters shown at the lower left of Figure 6. After passing through these two final filters, the wastewater was returned to the treatment plant.

Most virus collection occurs on the two final adsorbing filters. However, for those experiments requiring clarifier filters, the latter must also be subjected to virus removal procedures. These experiments have shown some viruses were adsorbed to clarifier filters.

Viruses were eluted from the filter surfaces using controlled methodologies which have been developed over several years at Carborundum as well as other research and governmental laboratories. These procedures included a saline rinse to remove residual chemicals and elution with 0.5 M glycine at a pH above 10. This step typically produced between 1 and 3 liters of solution containing most of the viruses present in the original wastewater sample.

Further liquid volume reduction was accomplished in the field using a gel reconcentration technique which incorporated 3 mM $AlCl_3$ coupled with Na_2CO_3 forming a floc at pH 7 and gravimetrically or centrifugally separating the supernatant. Viruses were contained in the floc. Viruses were removed from the floc by raising the pH above 10 with 0.5 M glycine, collecting it, neutralizing, and adding 10% fetal calf serum. The final concentrate was quickly frozen at dry ice temperatures and shipped to our assay laboratory. Shipping was in accordance with all Federal, State, and Carrier regulations.

Special care was taken throughout this process to eliminate sample contamination by either the sample collector, the surroundings, the containers, or shipping techniques.

Laboratory Techniques

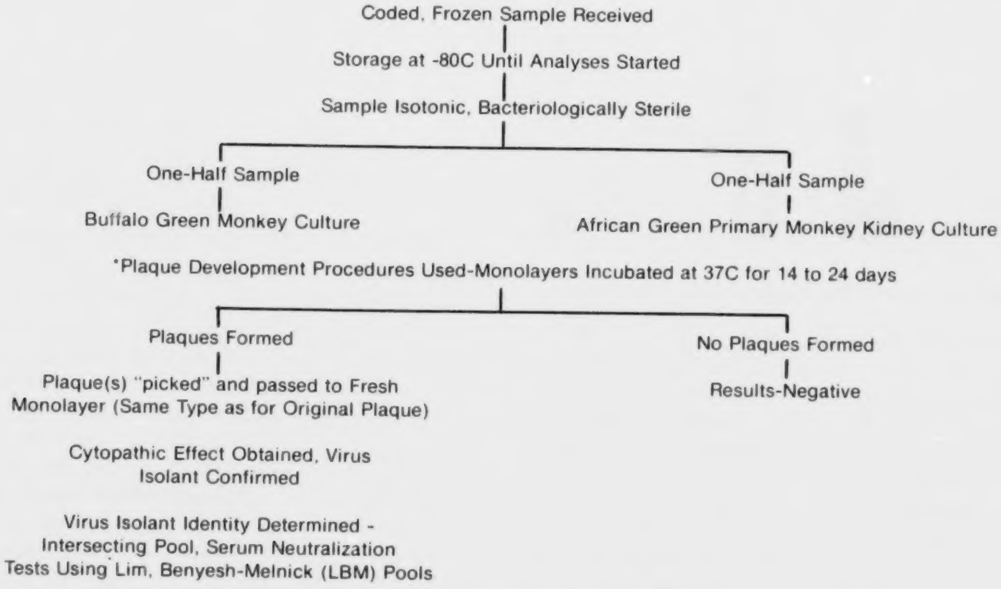
Samples collected during this program were analyzed for virus concentration and identification, under Carborundum's direction, in Durham, New Hampshire.

A schematic diagram (Figure 7) depicts the route taken by each sample within the laboratory.

Until it was placed on test, each sample was maintained at -80°C to avoid denaturation and other changes detrimental to virus recovery and infectivity. Removal of bacteria, yeast and mold was accomplished by either an 8-16 hour ether or an antibiotic treatment. Diethyl ether was used in our early experiments since enteroviruses have been shown to be ether resistant under these test conditions. However, advantages were identified by the autumn of 1977 in using antibiotics: gentamicin (50 ug/ml), penicillin (100 unit/ml), streptomycin (100 ug/ml), and fungazone (1 ug/ml). Hence, some changes in absolute sensitivity may have occurred. However, relative sensitivities remained unaffected.

Following sterilization treatment, samples were hydroextracted using polyethylene glycol. This procedure further concentrated the samples prior to test.

Natural virus testing was split between two culture types: Buffalo Green Monkey (BGM) and Primary Monkey Kidney - African Green (PMK). This was


Coded, Frozen Sample Received
Storage at -80C Until Analyses Started
Sample Isotonic, Bacteriologically Sterile
One-Half Sample
One-Half Sample
Buffalo Green Monkey Culture
African Green Primary Monkey Kidney Culture
*Plaque Development Procedures Used-Monolayers Incubated at 37C for 14 to 24 days
Plaques Formed
No Plaques Formed
Plaque(s) "picked" and passed to Fresh Monolayer (Same Type as for Original Plaque)
Results-Negative
Cytopathic Effect Obtained, Virus Isolant Confirmed
Virus Isolant Identity Determined - Intersecting Pool, Serum Neutralization Tests Using Lim, Benyesh-Melnick (LBM) Pools

*Controls for non-virus contaminants, adventitious virus, monolayer integrity-monolayers inoculated with sterile diluent and overlaid with overlay media

FIGURE 7. TEST PROCEDURES USED FOR VIRUS ANALYSES OF TEST SAMPLES

done to obtain as great an isolation sensitivity as possible. All examinations in cell cultures were made by means of plaquing methods which allowed separation and enumeration. Cell culture incubation periods of up to 24 days were used to maximize development of slow-forming plaques.

Following the formation of a plaque, natural viruses were recovered by means of "plaque picks". This involved transferring the picked plaques to a fresh culture monolayer with a liquid overlay medium. The monolayer was always of the same type as that used for the initial recovery. Consequently, the loss of virus during passage due to the use of a nonsensitive culture was avoided. Identification of isolates was made by intersecting pooled serum-neutralization tests carried out in microtiter plates (Lim-Benyesh-Melnick Antisera Pools).

RESULTS

Virus Isolation Rates and Concentrations

Virus isolation rates for each of the five wastewater treatment plants are given in Figure 8. The white area represents the isolation rates over the test period shown. The isolation rate is calculated for both the treated and treated plus disinfected wastewater by equally weighting each sample shown to contain one or more virions. At least forty percent of all samples of treated wastewater contained at least one virion prior to the disinfection stage of treatment.

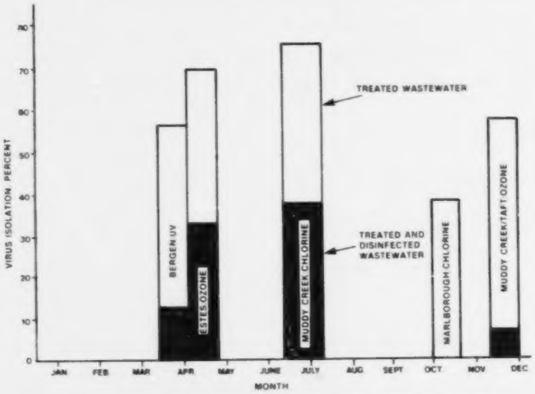


FIGURE 8. MONTHLY VIRUS ISOLATION RATE

These results showed no major differences between seasonal isolation rates, particularly for the two sampling periods completed in July and December at the Muddy Creek Sewage Works. These two rates were 75% and 60%, respectively, and to identify significant differences would require more testing. This finding may be significant in the disinfectant usage plans being proposed for winters in the Northeast of the U.S.

The virus isolation rates shown as blackened-in areas under each treated wastewater level indicator show that significant virus reductions were measured. Reductions occurred for each of the three disinfectants (UV, chlorine, and ozone) studied. Virus isolation rates were reduced four times for UV; 29 times for ozone (Estes Park); for chlorine, two times in one case, and infinity in the second; and for packed column ozone (Taft), eight times. Additional sampling would be required to obtain a definitive difference between the three methods of disinfection.

These same data were analyzed to obtain an average virus concentration per one hundred gallons of both treated and treated plus disinfected sewage. All samples taken at each plant were analyzed and the results averaged. The results represent the average concentrations for about 300 gallons of effluent sampled at each site. Figure 9 is a comparison of these average concentrations. Again, there may be no significant seasonal differences, especially when comparing the two Muddy Creek Sewage Works data for July and December. Both sets of data show about six virions per one hundred gallons of treated wastewater.

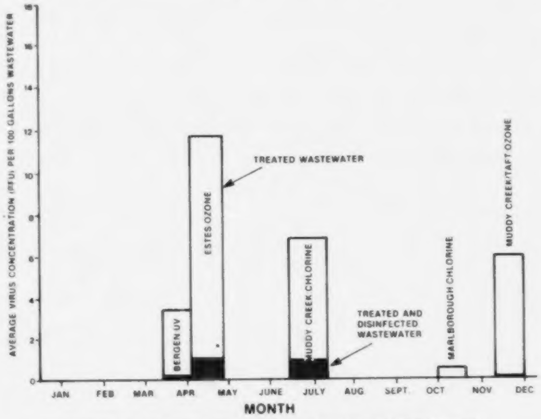


FIGURE 9. MONTHLY VIRUS CONCENTRATION

The blackened-in area in Figure 9 shows the virus concentrations after the disinfection step. The Estes Park Wastewater Treatment Plant had the poorest ratio of disinfected vs. treated wastewater of those plants measured, However, during four of the days we tested the plant, the ozone dosage rate dropped below 3 mg/l, and hence this was likely the cause of high virus concentrations in the effluent. Neglecting the data collected on those four days gave the blackened-in area shown in Figure 9. The average concentration would have been 8 pfu/100 gal. with all data included. A comparison of these isolation rates as a function of applied ozone dosage will be presented later.

Statistical Significance of Disinfection

Application of the Student's t Test to all of our measured virus isolation rates and concentrations is given in Table 2. The t-statistic is given along with its implied significance level. The smaller this value the more likely the two virion distributions (before and after administering a disinfectant) are significantly different. A low significance level is an indication that the two levels are significantly different and hence that the daily decreases due to disinfection are significant and can be attributed to the addition of UV, ozone, or chlorine.

TABLE 2. STUDENT'S t SIGNIFICANCE TEST FOR VIRUS IN TREATED VERSUS DISINFECTED SEWAGE

	Bergen (UV)	Estes (Ozone)	Muddy Creek (Chlorine)	Marlborough (Chlorine)	Muddy Creek/Taft (Ozone)
t-Statistic	2.5	(1.4)* 1.0	2.2	2.5	1.9
Average Discharge Conc./100 gal.	0.1	(0.4) 8.0	1.0	0	0.07
Mean of Difference Conc./100 gal.	1.1	(3.7) 3.6	6.1	0.6	6.1
Std. Dev. of Difference Conc./100 gal.	1.8	(8) 14	11	0.9	12
Significance Level	2.2%	(16%) 16%	2.4%	2.2%	3.9%

*Bracketed Values Neglecting Four Problem Days

The average discharge concentrations as well as the mean value of the differences in virus concentrates before and after disinfection are given in Table 1. The discharge concentrates range from "0" for chlorine added to nitrified sewage to 1.0 virion per 100 gal. for chlorine added to secondary treated sewage. A measure of virus reduction is obtained by dividing the mean concentration difference by the discharge concentration. These values range from about six for chlorine added to secondary treated sewage to infinity for chlorine added to denitrified sewage.

Virus Identification

A summation of all tests from this program to determine the virus identification distribution is shown in Figure 10. The relative virus concentrations based on total virus in the treated wastewater is presented. Polio and Coxsackieviruses were separated and identified. All three polioviruses (types 1, 2 and 3) were found in this study. However, it is not known whether they were the "wild" or vaccine types. The wild strain of poliovirus is known to produce meningitis and paralytic polio in man. Coxsackie A9, B1, B2, B3, B4, B5 and B6 were also separated and identified during this program. These latter viruses cause summer rash, polio-like paralysis, chest pains and colds. Note that, except for the CB5 virus, all virus types were found in

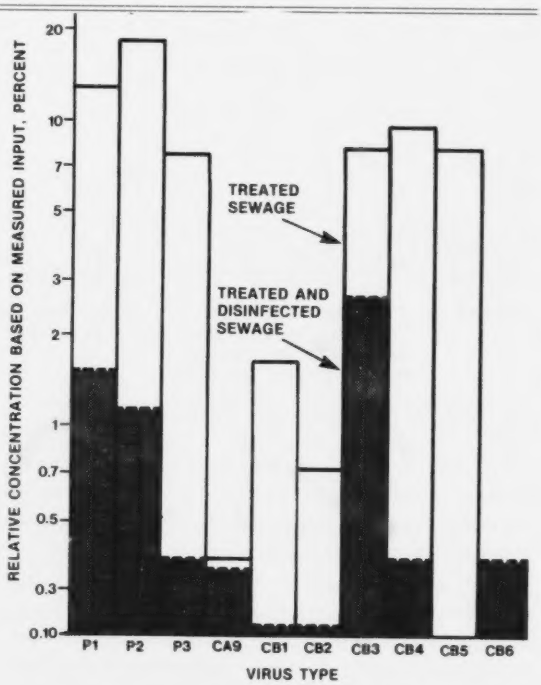


FIGURE 10. VIRUS IDENTIFICATION: BOTH BEFORE AND AFTER DISINFECTION WITH CHLORINE, OZONE, OR UV LIGHT.

the treated as well as in the treated plus disinfected wastewater effluent.

Diurnal Variations

Virus concentrations in treated wastewater can vary as a function of time, as shown in Figures 11 and 12.

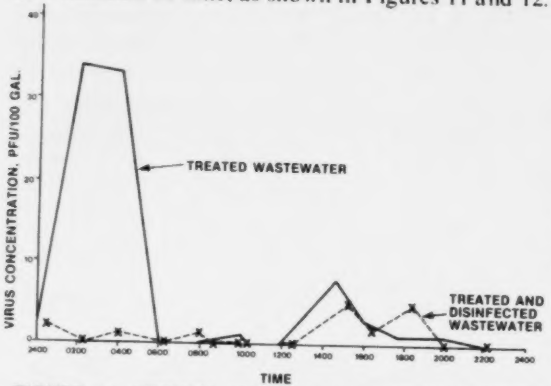


FIGURE 11. DIURNAL VARIATION OF VIRUS CONCENTRATIONS IN WASTEWATER EFFLUENTS DISINFECTANT: CHLORINE MUDDY CREEK: DECEMBER 1976

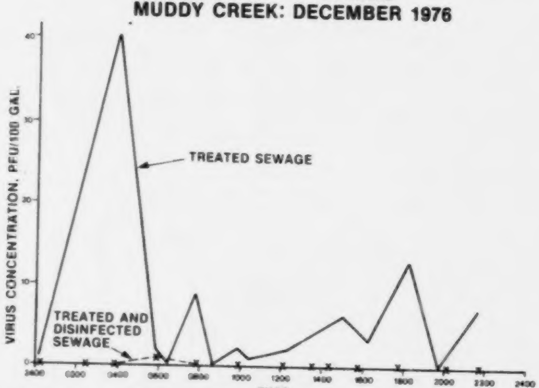


FIGURE 12. DIURNAL VARIATION OF VIRUS CONCENTRATIONS IN WASTEWATER EFFLUENTS DISINFECTANT: OZONE MUDDY CREEK: JULY 1977

The Muddy Creek Sewage Works was sampled to study these diurnal variations. Virus concentrations were significantly higher (as high as 40 pfu per 100 gallons) between 2:00 AM and 4:00 AM in both the December and July studies. An increase of almost an order of magnitude is evident.

This increase remains unexplained at this time. Perhaps the ultraviolet light from the sun may be a factor in virus inactivation during daylight hours. No comparison is available for other plants since this was the only plant we visited where staggered sampling was conducted for all wastewater quality measurements.

The increased virus challenge to the plant between 2:00 AM and 4:00 AM did not result in a significant increase in measured virus levels in the effluents during this same period. The dashed lines indicate virus concentrations for treated disinfected effluent collected during the same time period as the treated effluent. Measurement of the virus challenge to a wastewater treatment plant should take into consideration the possibility of diurnal variations.

Virus Indicators

The diurnal variation of effluent virus levels demonstrated at the Muddy Creek Treatment Plant provided a unique opportunity to study the correlation between the virus levels and the normal wastewater quality parameters.

The sizable variation in these virus concentrations would signify that changes in virus concentrations might be correlated with changes in other parameters, especially if attempts are made to use these parameters as virus indicators.

Results of regression tests using virus concentration as the dependent variable against six separate independent variables in the treated effluent from Muddy Creek during July 1977, are summarized in Figure 13.

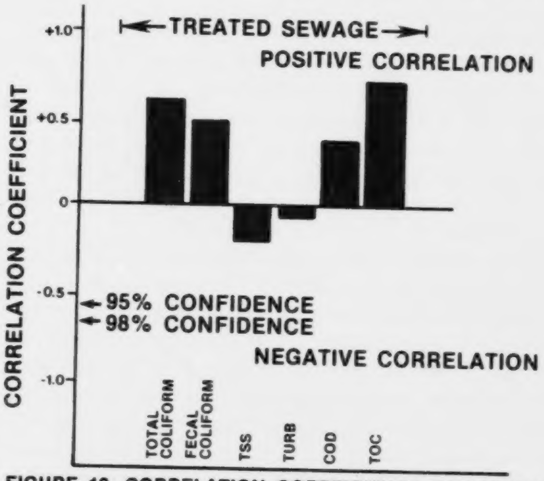


FIGURE 13. CORRELATION COEFFICIENTS FOR VIRUS IN-TREATED SEWAGE: MUDDY CREEK: JULY

Total coliform, fecal coliform, chemical oxygen demand, and total organic carbon had a positive correlation coefficient. The lack of correlation for total suspended solids and turbidity was surprising, since many researchers believe that viruses can attach to particulates and thus are more likely to survive various treatment modes. However, the confidence level was sufficiently low to make any conclusions highly speculative.

A similar analysis of wastewater quality data collected in December is shown in Figure 14. Here, no correlation was found for total coliform and fecal coliforms while total suspended solids and turbidity had a positive correlation coefficient. Both COD and TOC exhibited a positive correlation in December as well as July.

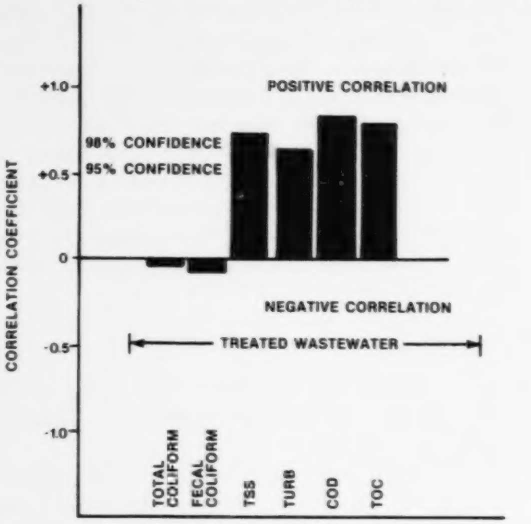


FIGURE 14. CORRELATION COEFFICIENTS FOR VIRUS IN TREATED WASTEWATER: MUDDY CREEK DECEMBER

The picture gets even more confusing when looking at treated, disinfected wastewater. Figure 15 presents the calculated correlation coefficients for disinfected wastewater at Muddy Creek in July. Here the values are not only mixed, but much too low for reasonable significance.

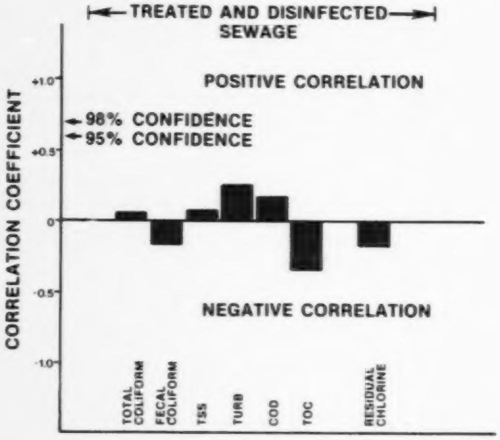


FIGURE 15. CORRELATION COEFFICIENTS FOR VIRUS IN TREATED, DISINFECTED WASTEWATER: MUDDY CREEK JULY

Coliform Studies

A comparison of total coliforms and fecal coliforms at each plant site visited during the two year study is shown in Figure 16. Treated wastewater had measured coliform concentrations between 10^4 coliforms/100 ml (Marlborough) and almost 10^6 coliforms/100 ml (Estes).

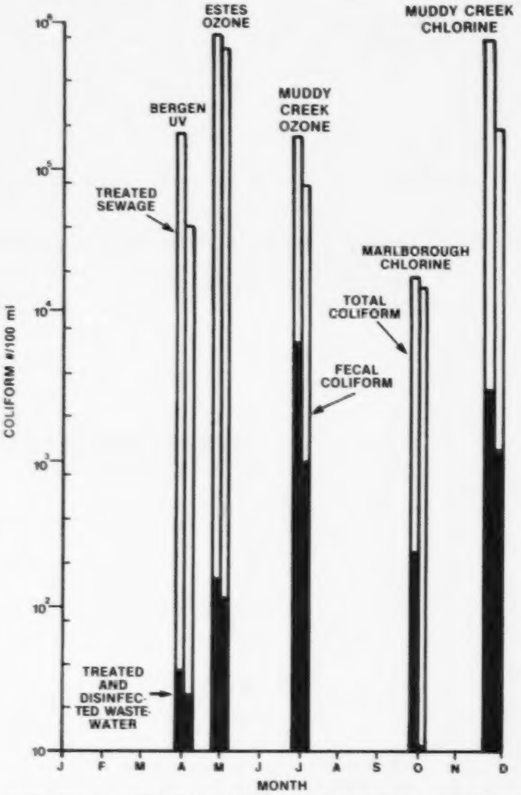


FIGURE 16. MONTHLY TOTAL AND FECAL COLIFORM CONCENTRATIONS

As with the virus data, there is no indication that summer months had higher coliform concentrations than other times of the year. In particular, the Muddy Creek data showed a slight increase in total and fecal coliform counts in December as compared to July.

The blackened-in fraction of the data is a measure of the coliform count after disinfection. Again, Muddy Creek data indicate ozone and chlorine were similar in disinfection effectiveness. UV light was highly effective for both total and fecal coliform reduction.

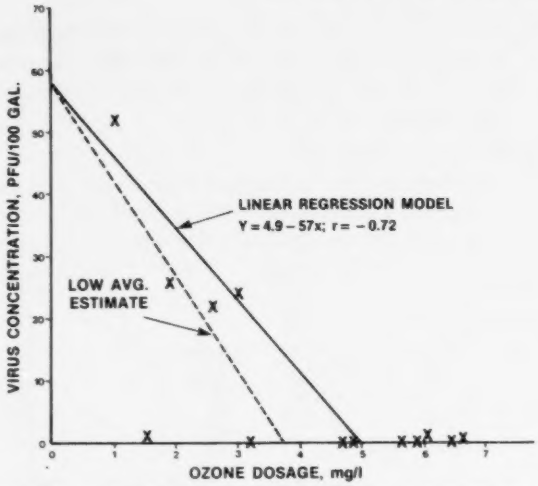


FIGURE 17. VIRUS CONCENTRATION AS A FUNCTION OF OZONE DOSAGE
ESTES PARK, COLORADO
MAY, 1977

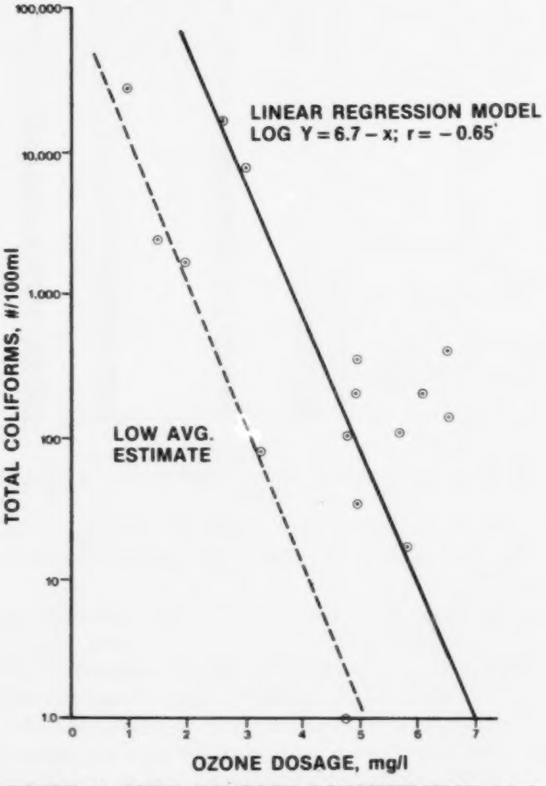


FIGURE 18. TOTAL COLIFORM CONCENTRATION AS A FUNCTION OF OZONE DOSAGE
ESTES PARK, COLORADO
MAY, 1977

Ozone as a Disinfectant

As indicated earlier, during part of our sampling program at Estes Park Wastewater Treatment Plant, the ozone dosage was inadvertently reduced on four of the sampling days. This enabled evaluation of microorganism reduction as a function of the applied dosage. Figure 17 shows virus data and Figure 18 shows total coliform data as a function of ozone dosage. The correlation coefficient in both cases is moderately high: -0.72 and -0.65, respectively. The resulting regression equations indicate that a dosage of greater than 4.5 mg/l was required to disinfect the effluent leaving this plant.

SUMMARY

In summary, no correlation was found between virus concentrations and any of the typical wastewater quality parameters.

Survival of viruses and total coliforms showed a strong negative correlation to applied ozone dosages between 1 and 7 mg/l. Dosages above 4 to 5 mg/l were required to ensure virus die-off.

No real seasonal variation of virus was detected. However, a strong diurnal variation was found at one plant. Maximum virus concentrations occurred between 2:00 and 4:00 AM.

Finally, at least ten different types of viruses were found in the wastewater effluents tested.

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24.

EFFECTS OF CHLORINE, OZONE, AND ULTRAVIOLET LIGHT ON NONVOLATILE ORGANICS IN WASTEWATER EFFLUENTS

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INTRODUCTION

The severity of water-borne diseases and disease epidemics has been greatly reduced because of disinfection treatment of wastewater and potable waters. During the last half century, the principal disinfectant has been chlorine. It was only recently established that chloro-organics are formed when chlorine is used to disinfect wastewater effluents and potable waters (3, 7, 8, 10, 12, 13). These findings are attributable to the development of improved and more sensitive analytical methodologies in the last decade. Because of the possible harmful effects of exposure to chlorinated organics (9), there is considerable interest in the use of possible alternatives to chlorine; currently, the principal alternative is considered to be ozone (6, 9). The chemistry of ozone reactions with specific organic chemicals has been studied extensively. However, the effects of ozonation on wastewater and wastewater treatment plant effluents have been characterized principally in general terms such as oxidation of organic carbon, biochemical oxygen demand (BOD), and color (9). Relatively little is known about the formation or possible formation of ozonation and ozonolysis products during disinfection with ozone, although the first products of ozone oxidation of organic molecules are presumed to be oxygenated materials that are water soluble and not highly volatile (9). Additional information is needed before ozone can be evaluated as an alternative to chlorine. Another alternate disinfection method is ultraviolet (UV) light. If this method becomes a viable alternative, irradiation effects on organic constituents in wastewater must be studied.

Rapid progress has been made recently in the analysis of volatile organic constituents in waters of environmental concern. However, the determination and characterization of organic constituents of relatively low volatility have not advanced as rapidly even though the greatest portion of dissolved organic carbon in wastewater and natural water consists of compounds in this category (4, 11). This lack of amenability to analytical techniques may be due to the inherently greater complexity of the "nonvolatile" molecular constituents (4). Nevertheless, a complete understanding of possible environmental and health-related effects which may be attributable to the nonvolatile organic constituents can only be based on definitive data about the chemical nature of the nonvolatile molecular constituents.

This paper reports the current experimental results of the effects of disinfection with chlorine, ozone, or UV light on the nonvolatile organic constituents in municipal wastewater treatment plant effluents. Results of mutagenic activity tests on effluent concentrates will be presented in a corollary paper in these proceedings (5).

ANALYTICAL METHODS

The basic analytical steps used to examine the effluent samples are 1) concentration, 2) separation of constituents, and 3) identification of constituents.

Concentration

The lower limit of detection for the high-resolution liquid chromatographs is in the microgram range, depending on the UV absorption of the individual compound. Because the high-resolution chromatographs are limited to 5 ml or less per sample, and the concentrations of specific contaminants in effluent samples may be 10μ g/liter or less, it is necessary to concentrate wastewater effluents by factors $\leq$3000 prior to analysis. The method of low-temperature distillation followed by two freeze-drying steps, as shown in Fig. 1, was selected in previous studies as the most convenient and suitable concentration method and provided adequate recovery of stable, nonvolatile organic compounds (10). However, during this investigation we have determined that lyophilization can be used for the vacuum distillation step and effect a savings in both personnel time and effort.

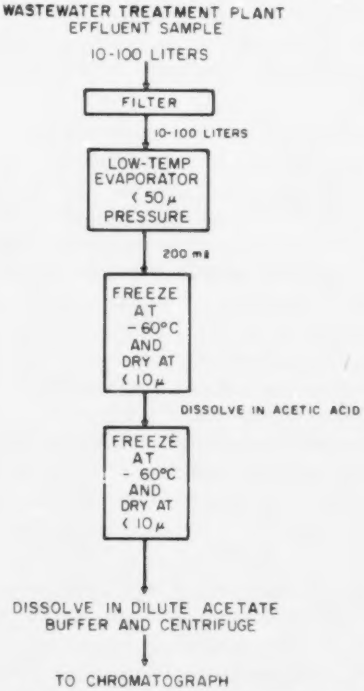

FIGURE 1. Procedure for concentrating effluent samples.

Separation of Constituents

Liquid chromatography has proved useful for separation and identification of numerous constituents in wastewater effluents. In a previous high-pressure liquid chromatography (HPLC) study of primary and secondary effluents from municipal wastewater treatment plants, approximately 60 organic compounds were identified and 100 additional compounds were characterized with respect to mass spectra and gas chromatographic properties (10).

In that study, high-resolution anion exchange chromatographs were demonstrated to have sensitivity in the microgram range and were capable of detecting and quantifying many individual organic compounds in concentrates of the complex aqueous effluent samples.

Both a preparative-scale and analytical-scale chromatograph (Figure 2) are used to separate and detect UV-absorbing compounds and/or cerate oxidizable compounds (e.g., carbohydrates, organic acids, phenols). The chromatographs consist primarily of a heated, high-pressure ion exchange column; a sample injection valve; a two-wavelength dual-beam UV photometer; a cerate oxidative monitor; and a strip-chart recorder. The ion exchange column for each system is a 50cm length of type 316 seamless stainless steel tubing (0.45 to 1.0cm ID), usually packed with strongly basic anion exchange resin. A 0.05 to 5.0ml sample (the volume depending on the inside diameter of the ion exchange column and the nature of the sample) is applied to the column by a six-port injection valve mounted as near to the top of the column as possible to minimize peak broadening (10).

The chromatograms are developed by eluting the sample constituents from the resin column with an ammonium acetate—acetic acid buffer solution (pH 4.4) whose acetate concentration gradually increases from 0.015 to 6.0 $\underline{M}$. The UV absorbances of the column effluent are measured at 254 and 280 nm using a dual-beam flow-through photometer and are recorded on a strip chart. Subsequently, the cerate oxidizability of the same effluent is measured and recorded.

The use of the cerate oxidative monitor in series with the UV photometer permits detection of additional compounds and a much greater sensitivity to other organic constituents such as phenols. It also provides another parameter useful in compound identification since many compounds that are UV absorbing are oxidizable. In this system, cerium (IV) is reduced to fluorescent cerium (III) by oxidizable compounds.

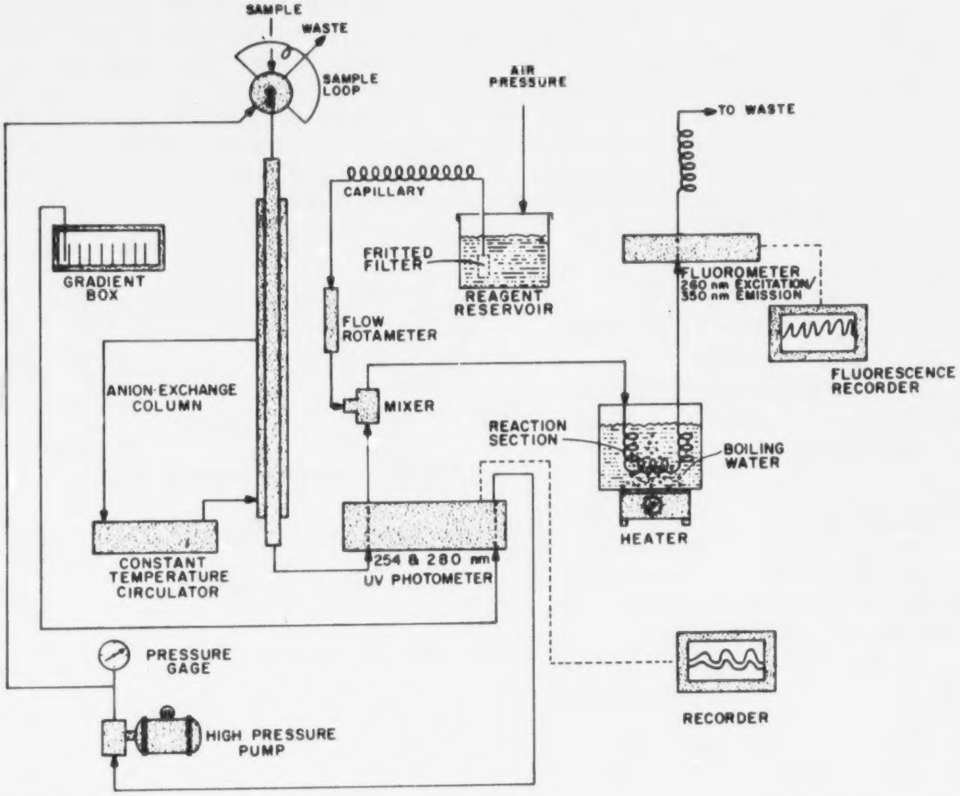

FIGURE 2. Schematic of HPLC system.

Identification of Constituents

The preparation of samples for analysis, the separation of constituents, and the application of analytical methods to separated fractions involve an integrated and complex series of manipulations and investigative techniques.

The preparative-scale liquid chromatograph system, which is coupled to a fraction collector, is capable of chromatographing 5ml of sample with a resolution approaching that of the analytical column. The eluate representing a component is collected and processed through the following analytical procedure to identify and characterize the isolated constituents.

Preparation of Fractions for Analyses. Eluted Fractions corresponding to individual chromatographic peaks from the anion exchange separations are frozen at -60° C and lyophilized for removal of the ammonium acetate—acetic acid buffer. The samples are then dissolved in spectroscopic-grade methanol.

Ultraviolet Spectrometry. For each of the collected fractions in methanol solution, UV spectra are obtained from 320 to 210 nm on a Beckman DB-G recording spectrophotometer and compared with UV spectra of reference compounds obtained in the same manner.

Gas Chromatography. Conversion of the nonvolatile constituents to volatile compounds is necessary for analysis by gas chromatography. The principal method of forming volatile derivatives of the nonvolatile compounds is silylation with bis(trimethylsilyl)-trifluoroacetamide (10). Although silylation is the preferred derivatization method, constituents may also be gas chromatographed as the methylated derivative. Methylation is accomplished with diazomethane in ether solution (14). The samples are analyzed by gas chromatography on one of several suitable instruments available in this laboratory. A variety of detection modes may be used, including flame ionization, electron capture, and electrolytic conductivity.

Mass Spectrometry. Mass spectrometry is usually performed on aliquots of the trimethylsilyl (TMS)-derivatized samples described above.

The TMS-derivatized sample is run routinely on a Finnigan model-3000 gas chromatograph—mass spectrometer. This provides data concerning the molecular weights of the constituents and the number of active hydrogen atoms per molecule. Comparison of the fragmentation pattern with that of reference standards is necessary for absolute identification.

DISINFECTION TREATMENT OF EFFLUENT SAMPLES

Chlorination

All chlorinated effluent samples were disinfected at the wastewater treatment plant during the routine chlorination of effluent.

Ozonation

Wastewater effluent samples are treated with ozone generated from a Wellsbach T-23 ozonator. A small fraction of the oxygen feed gas is converted to ozone which is allowed to react with the sample for a predetermined amount of time, depending on the total dose desired. The reaction takes place in a round-bottomed flask using a medium-porosity gas diffuser to bubble the ozone and the oxygen carrier gas through the magnetically stirred sample solution. Nonreacting ozone is trapped in a 20% KI solution during the reaction run; after the run, the headspace above the effluent sample is purged with air into the trap to determine the amount of unreacted ozone. A wet-test meter records all gas flows. A schematic drawing of the ozonation equipment is shown in Figure 3.

Ozone concentration in the gas is determined by bubbling the gas through a 20% KI solution and titrating with sodium thiosulfate in an acidic medium with starch indicator.

A typical reaction run might have a gas concentration of 25 mg 0_3/l gas and a gas flow of 0.25 l/min. With a 4.5-l sample and a reaction time of 8.64 min, a resultant sample concentration of 12 mg 0_3/l effluent would be produced.

Ultraviolet-Light Irradiation

Samples of primary or secondary wastewater effluents are exposed to varying dose levels of UV irradiation by forced flow through a UV sterilizer. A model H-1.0 UV sterilizer with sight port (Pure Water Systems, New Jersey) is used with an LFE centrifugal pump (Eastern Industries Div., 1 liter/min max). The internal volume of the baffled-flow cell is 370 ml. A quartz tube with a low-pressure mercury lamp, capable of emitting UV irradiation at 254 nm, is positioned longitudinally through the center of the cell. As the sample flows through the cell, it is exposed to a constant UV-irradiation dose level. The amount of energy impinging on the sample is calculated by multiplying the retention time in the cell by the measured optical irradiation intensity.

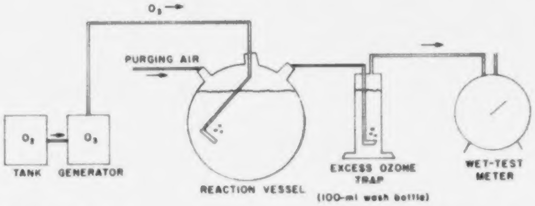

FIGURE 3. Schematic drawing of ozonation equipment.

Using the following equation,

$$\frac{V_c}{F} = t_R$$

where

V_c = volume of cell,
F = flow rate, and
t_R = retention time,

one can calculate the retention time of a sample for each flow rate. The t_R value times the optical irradiation (μ W/cm^2) measured with an IL-700 Research Radiometer and Photometer gives the amount of energy impinging on the sample during this time (μ W•sec/cm^2).

EXPERIMENTAL RESULTS

Ozonation

Initial efforts were directed toward establishing parameters for ozonation of secondary effluent from wastewater treatment plants. In the typical secondary effluent from the Oak Ridge East Wastewater Treatment Plant, an ozone dose of approximately 14mg of ozone per liter of effluent is required for a ≥99.99% bacterial kill (Figure 4) as established by the standard plate-count method for determination of bacteria (1). Initial determinations of the bacterial quality of the control and treated water samples were made using this method; however, to facilitate comparison with other disinfection studies, later determinations used the total coliform method (membrane filter) (2).

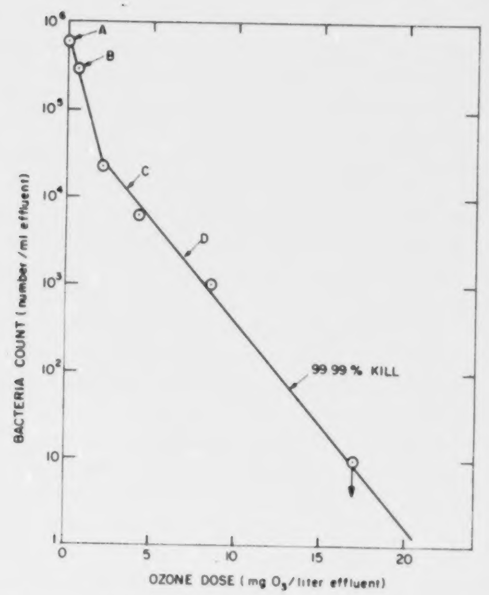

FIGURE 4. Disinfection of secondary effluent with ozone.

Large differences were detected between high-resolution anion exchange chromatograms of concentrates of unozonated wastewater effluent and those that had been ozonated for 3, 15, and 30-min periods with an ozone gas concentration of 6 mg per liter of oxygen gas. This treatment resulted in total doses of 0.7, 3.5, and 6.9 mg of ozone per liter of wastewater effluent. The chromatograms of the UV-absorbing and the cerate-oxidizable constituents in the control sample and the ozonated samples are shown in Figures 5 and 6. The ozone dosages of these samples are labeled A, B, C, and D on Figure 4 for comparative purposes to show approximate levels of disinfection, although bacterial counts were not made for these samples.

It is apparent that ozone initially reacts with organic constituents that are probably of a higher molecular weight and are not separated in this chromatographic procedure, although some may be associated with the first chromatographic peak. The higher-molecular-weight constituents are broken down into smaller-molecular weight fragments that are separated chrom-

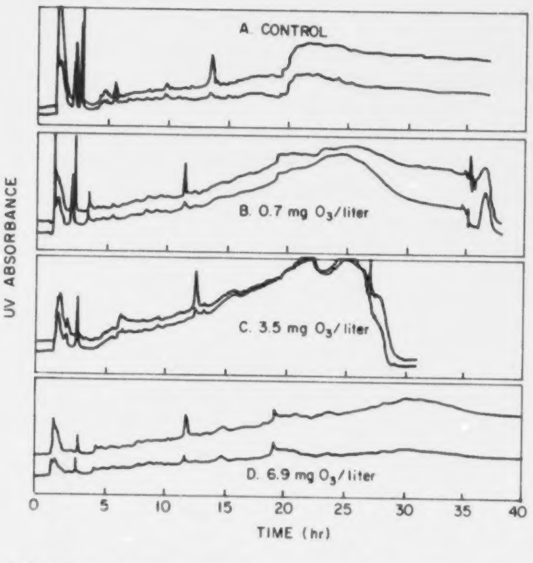

FIGURE 5. Comparison of UV-absorbing constituents in secondary effluent sample with samples treated with ozone. Sample A is the unozonated control. Samples B, C, and D were ozonated at ozone doses of 0.7, 3.5, and 6.9 mg/liter respectively. Chromatograms were determined for 1.0-ml aliquots of 2500-fold concentrates using standard UV-Analyzer chromatographic procedure. For each chromatogram, the bottom tracing represents the UV absorbance at 254 nm.

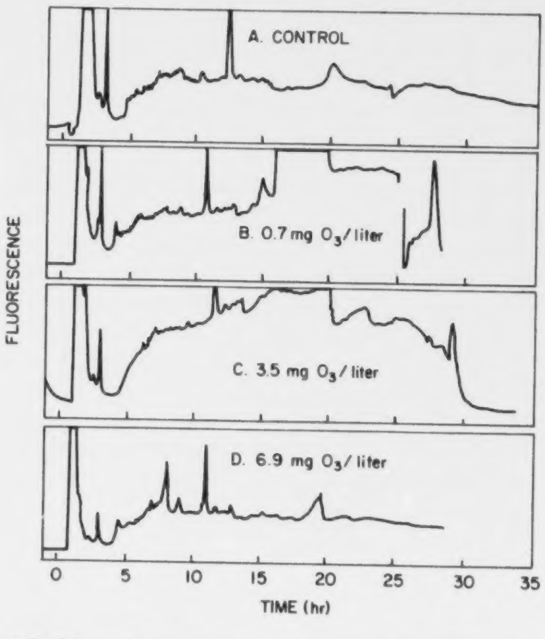

FIGURE 6. Comparison of oxidizable constituents in a secondary effluent sample with samples treated with ozone. Sample A is the unozonated control. Samples B, C, and D were ozonated with ozone doses of 0.7, 3.5, and 6.9 mg/liter respectively. Chromatograms were determined for 1.0-ml aliquots of 2500-fold concentrates using standard cerate-oxidimetry chromatographic procedure.

atographically and detected as UV-absorbing and oxidizable peaks. This is indicated by a decrease in size of the first peak (Figures 5 and 6) with increasing ozone dosage and an increase of poorly resolved peaks in the middle of the chromatograms for the intermediate dosages. As ozonation proceeds, these fragments are, in turn, oxidized and decomposed as shown in Figures 5 and 6 for the highest dosage.

The reaction conditions, chemical data, and disinfection data of the ozonation experiments used to calculate the kill curve in Figure 4 are given in Table 1. The chemical oxygen demand (COD) and BOD of the samples trend downward with increasing dose. The total organic carbon (TOC) apparently increases slightly, then decreases with increasing dose. No conclusion can be drawn regarding total Kjeldahl nitrogen (TKN).

Figure 7 shows the chromatograms of UV-absorbing and cerate oxidizable constituents in another ozonation experiment with secondary effluent from the Oak Ridge East Wastewater Treatment Plant. Two 6-liter aliquots of Oak Ridge East Wastewater Plant secondary effluent were ozonated in the laboratory to dosages of 3.5 and 20.0 mg ozone per liter of wastewater. Reaction conditions were: 25° C; ozone gas concentration, 19.4 mg ozone per liter gas (oxygen); 12 liter glass reaction flask; magnetic stirrer; porous glass-frit gas diffuser, ozone-oxygen gas mixture flow rate of 250 ml/min.

From the chromatograms of the UV-absorbing constituents, it may be deduced that most of the destruction of UV-absorbing compounds occurs rapidly and within the first 3.5 mg/liter dosage of ozone. With respect to the oxidizable constituents, there is an apparent initial destruction of some compounds and an increase in numbers and concentration of others with increasing ozone dosage; for example, the increase in numbers and size of peaks in the 5 to 15 hr elution period. Chemical data for the control and ozonated wastewater samples are given in Table 2. Note that the disinfection data were not run. The usual ozone dosage for disinfection is 6 to 10 mg ozone per liter effluent; thus the chemical effects of ozonation for disinfection may be considered intermediate between the two laboratory-ozonated samples. The wastewater samples are representative of effluent from an activated-sludge secondary treatment plant. The wastewater was principally domestic, containing very little industrial waste.

In another experiment, a 20 liter sample of Oak Ridge West Wastewater Treatment Plant primary effluent was ozonated in two 10.5 liter batches to a dosage of 6.9mg ozone/liter effluent. The ozone-oxygen gas flow rate was 0.11 liter/min, and the total reaction time was 29.25 min. The system was purged with approximately 1 liter of air after each ozonation. In batch one, the total amount of ozone applied was 72.4mg; 26.5mg ozone was trapped in the KI solution following the reaction train, for an ozone utilization efficiency of 63%. In the second batch, the total amount of ozone applied was 72.4mg; 29.6mg ozone was measured in the KI solution following the reaction train, indicating an ozone utilization or a reaction efficiency of 59%. Thus, an average of 61% of the ozone dose reacted with the primary effluent in our experimental equipment. The chromatograms of UV-absorbing constituents in the untreated (control) sample and those in the ozonated primary effluent

TABLE 1. OZONATION OF SECONDARY EFFLUENT FROM THE OAK RIDGE EAST WASTEWATER TREATMENT PLANT: CHEMICAL AND DISINFECTION DATA (RUN 1)

Reaction time (min)[(a)]	Total ozone dose (mg/liter of effluent)	Total[(b)] Bacteria count (No./ml)	pH	COD	BOD$_5$	TKN	TOC
				mg/liter			
Control	0.0	600,000	7.5	24	9	20.0	7.9
5	0.7	300,000	7.8	26	9	18.6	7.6
15	2.1	22,500	7.8	20	7	18.6	8.3
30	4.3	6,100	7.9	19	<5	19.0	8.6
60	8.5	1,000	7.9	20	<5	23.5	7.5
120	17.1	<1	7.9	16	<5	18.2	7.2

(a) Room temperature (approximately 25° C); ozone-oxygen gas mixture at 0.08 liter/min containing 8.9-mg ozone per liter gas. 5-liter effluent sample, glass-frit sparger with magnetic stirrer.

(b) Standard plate-count method (1).

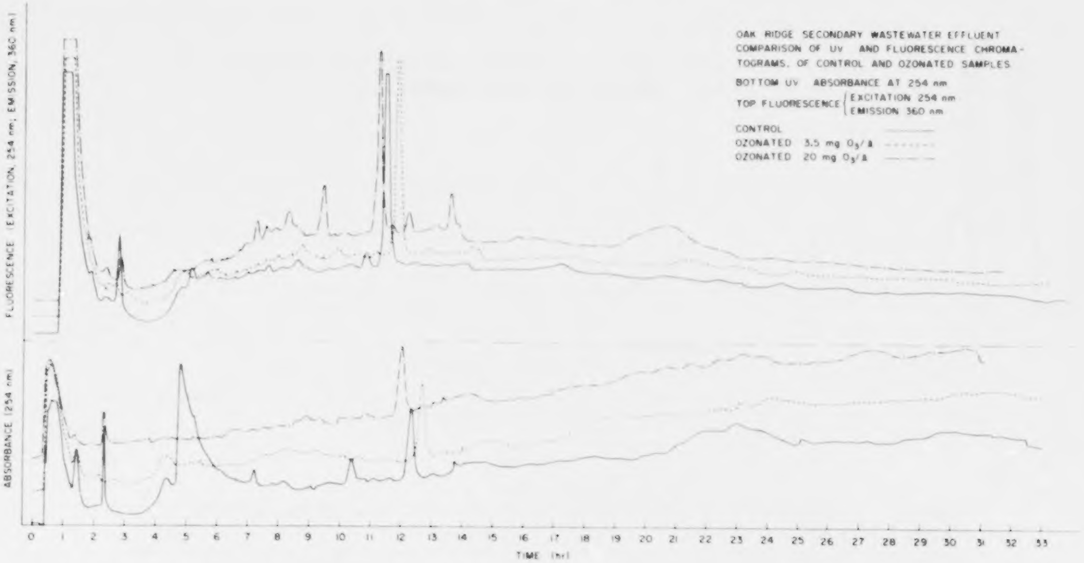

FIGURE 7. Chromatograms of UV-absorbing constituents and cerate-oxidizable constituents in secondary effluent ozonated with 0-, 3.5-, and 20-mg 0_3/liter effluent.

TABLE 2. OZONATION OF THE SECONDARY EFFLUENT FROM THE OAK RIDGE EAST WASTEWATER TREATMENT PLANT: CHEMICAL DATA (RUN 2).

Reaction time (min)(a)	Total ozone dose (mg ozone/liter effluent)	pH	COD	BOD_5	TKN	TOC
			mg/liter			
Control	0.0	7.61	30	6	18.2	10.8
4.3	3.5	7.69	28	7	17.4	9.8
24.8	20.0	7.84	24	5	17.2	10.5

(a) Room temperature (about 25° C): ozone-oxygen mixture at 250 ml/min containing 19.4-mg ozone/liter gas: 6-liter effluent sample, glass-frit sparger with magnetic stirrer.

sample are compared in Figure 8. It is evident that many of the UV-absorbing constituents are destroyed by ozonation. Note, however, the formation of one major peak eluting to approximately seven hours.

Ultraviolet Irradiation

Since UV irradiation is being evaluated as a potentially viable alternative disinfection process, studies are underway to determine its treatment effects on the nonvolatile organic fraction of wastewater effluents. This study involves irradiating secondary wastewater effluent at a fixed optical irradiance ($\mu W/cm^2$) at different flow rates (i.e., different retention times within the UV-flow cell).

Total coliform [membrane-filter (MF) method (2)] survival curves were determined for UV irradiation of primary and secondary effluents from the Oak Ridge Wastewater Treatment Plants (Figure 9). For the non-irradiated primary effluent, the BOD_5 was 80 mg/liter and the pH was 7.5. The non-irradiated secondary effluent BOD_5 was 19 mg/l and the pH was 7.1. The survival curves were used to establish the parameters for subsequent irradiation experiments.

The chemical effects of UV-irradiation disinfection were evaluated in a series of HPLC separations. Samples of unchlorinated secondary effluent were collected in 23-liter polyethylene carboys at the Oak Ridge East Wastewater Treatment Plant, which were delivered immediately to the laboratory with no refrigeration. Ambient temperature during collection and transportation was approximately 0° C. All samples

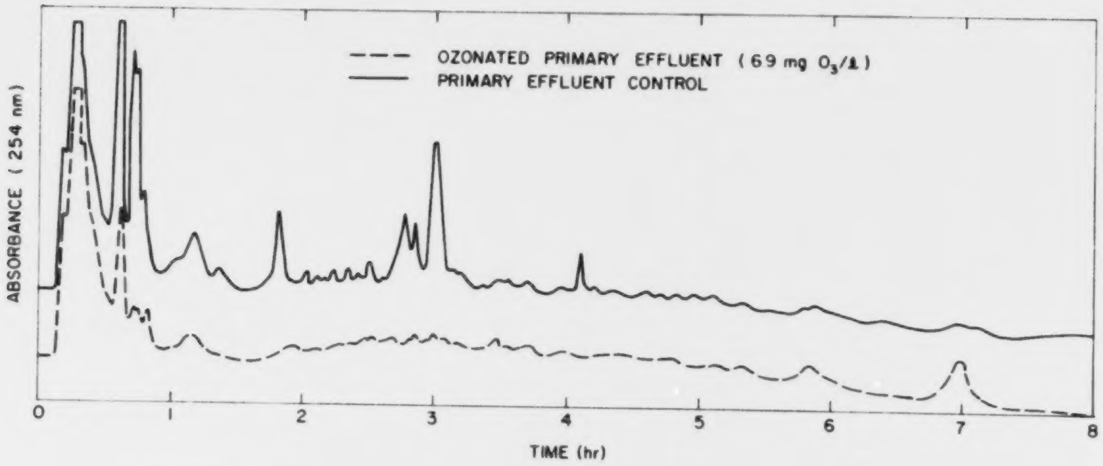

FIGURE 8. Chromatograms of UV-absorbing constituents in ozonated primary effluent (ozone dosage, 6.9 mg/liter) compared with undisinfected control primary effluent. Samples were collected from the Oak Ridge West Wastewater Treatment Plant and the ozonated sample was ozonated in the laboratory. Ozonated sample, 0.77 ml of 790X concentrate. Unozonated sample, 0.77 ml of 700X concentrate.

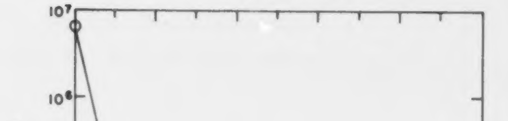

FIGURE 9. Disinfection of primary and secondary effluent with UV irradiation. Total coliform measurements were made using the membrane-filter method (2).

were filtered through a Millipore filtering system utilizing a prefilter and an 8μm Millipore filter. Filtered samples were then pumped through the UV sterilizer at different flow rates to vary retention time within the cell, and thus the irradiation level. A 20 liter aliquot from each irradiation level was lyophilized, concentrated 2000X, and compared by HPLC.

Chemical and disinfection data for the control and UV-irradiated samples are given in Table 3. It is interesting to note the apparent decrease in TOC and TKN with irradiation dosage increase. However, as indicated by the chromatograms of UV-absorbing compounds in the effluent concentrates (Figure 10), UV irradiation seems to have little chemical effect on the UV-absorbing constituents.

Comparison of Chlorination, Ozonation, and UV-Irradiation Effects

Sixty liters of unchlorinated secondary effluent and 20 liters of chlorinated effluent (2 mg/l chlorine dose; 0.5 mg/l chlorine residual) were collected from the Oak Ridge East Wastewater Treatment Plant. The secondary effluent samples were filtered through Whatman No. 2 filter paper. A 20 liter aliquot of the unchlorinated effluent was lyophilized and concentrated as the control sample. A second 20 liter aliquot was ozonated in two 10 liter batches at 25°C to an ozone dose of 8.0mg ozone per liter of effluent. Reaction conditions were: 20.0mg 0_3 per liter of oxygen

TABLE 3. EFFLUENT DATA FOR UV-IRRADIATION EXPERIMENT.

Sample	Concentration factor	UV-irradiation exposure (μW•sec/cm^2)	Total coliform[(a)] (No./100 ml)	pH	BOD$_5$	COD	TOC	TKN
					mg/liter			
1	1905	Control (0)	200,000	7.4	15	25	20	4.0
2	2000	13,300	<1	7.3	<5	24	20	3.7
3	1818	18,700	<1	7.4	<5	23	19	3.6
4	1800	39,400	40(b)	7.4	<5	22	16	3.3
5	1667	61,200	<1	7.3	<5	23	14	3.3

(a) Membrane-filter method (2).
(b) This may represent possible contamination.

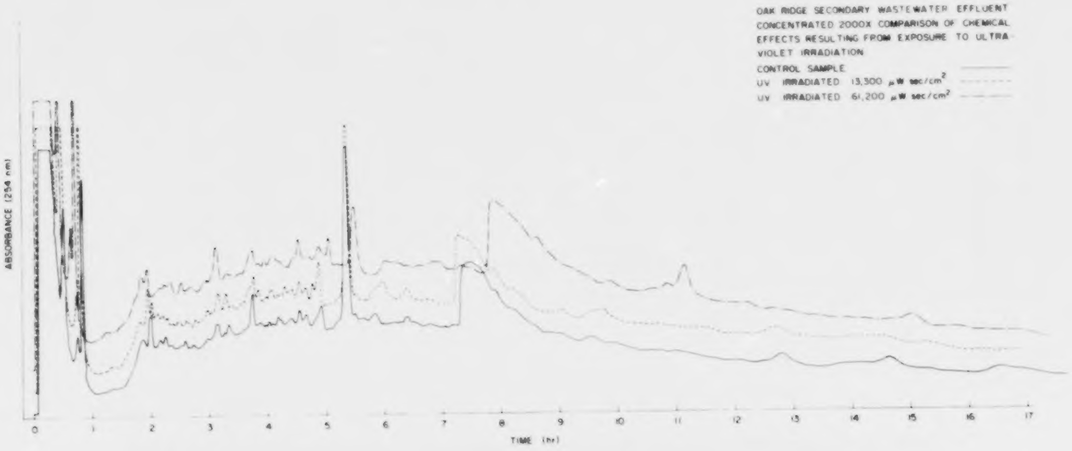

FIGURE 10. Comparison of chromatograms of UV-absorbing constituents in secondary wastewater effluents irradiated with UV light at 0, 13,300, and 61,200 μW•sec/cm^2.

gas; gas flow rate, 180 ml/min; reaction, 24.3 min; 12 liter glass flask; and magnetic stirrer. The ozone utilization efficiency in the reactor was 76%. After ozonation the sample was concentrated for HPLC. A third 20 liter aliquot was UV irradiated at 21,200 μW•sec/cm^2 at 25° C and concentrated. The 20 liter chlorinated aliquot was also concentrated by lyophilization for HPLC.

The chemical analyses and disinfection results for all samples are shown in Table 4. Chromatograms of UV-absorbing constituents and cerate oxidizable constituents are shown on Figures 11 and 12. Confirming previous effects, ozonation at disinfection concentrations destroys many nonvolatile organic constituents and produces other nonvolatile organics, especially oxidizable components. Ultraviolet irradiation apparently had little effect on both nonvolatile oxidizable organic constituents and the nonvolatile UV-absorbing components. As also evidenced in previous experiments, chlorination destroys some nonvolatile UV-absorbing and oxidizable constituents and forms several other constituents.

Similarly, effects of disinfection on the primary effluent were studied. Samples of both chlorinated and unchlorinated primary effluent were collected in 23 liter carboys at the Oak Ridge West Wastewater Treatment Plant and delivered to the laboratory without refrigeration. Ambient temperature during collection and transportation was approximately 1° C. All samples were filtered with a Millipore filtering system using an 8 μm filter.

Twenty liters of filtered, unchlorinated primary effluent were exposed at 27,500 μW•sec/cm^2 of UV irradiation. A second 20 liter sample was ozonated in two 10.5 liter batches to a concentration of 6.9mg ozone/liter effluent. The experimental details of this

TABLE 4. COMPARATIVE DISINFECTION OF A SECONDARY WASTEWATER EFFLUENT SAMPLE: CHEMICAL AND DISINFECTION DATA

Disinfection treatment	Treatment dosage or residual	Bacteria (a) count (No./ml)	pH	COD	BOD	TKN	TOC
				mg/liter			
Control	None	277,000	7.3	45	17	3.8	11.5
Ozonated	8 mg O_3/l effluent	500	7.5	40	7	8.2	7.0
Ultraviolet	21,200 µW•sec/cm²	100	7.4	39	<5	7.9	9.6
Chlorinated	0.5 mg/l residual	2,200	7.3	44	<5	9.2	8.9

(a) Standard plate-count method (13).

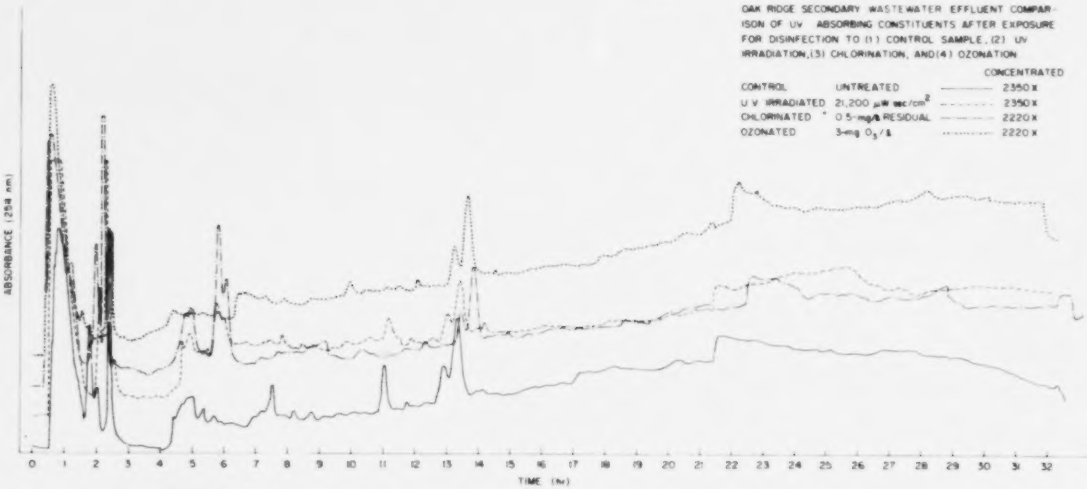

FIGURE 11. Comparison of chromatograms of UV-absorbing constituents in secondary wastewater effluents disinfected with chlorine, ozone, and UV light.

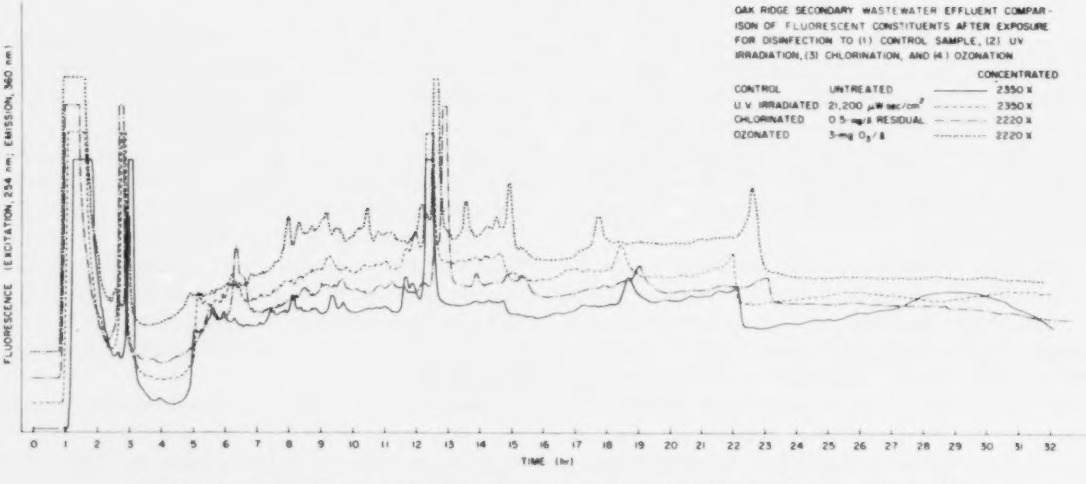

FIGURE 12. Comparison of chromatograms of cerate-oxidizable constituents in secondary effluent disinfected with chlorine, ozone, and UV light.

ozonation were reported in the section on ozonation.

Chemical and disinfection data for the four 20 liter aliquots are given in Table 5. The aliquots were concentrated and chromatographed. The chromatographic comparison of the chemical effects of chlorination, ozonation, and UV irradiation on the UV-absorbing constituents in primary effluent revealed significant chemical differences among the three disinfectants, as shown in Figures 8, 13, and 14. In these figures, the chromatogram for the UV-absorbing constituents in the control samples are offset above the chromatograms of the disinfected samples for comparison purposes. In Figure 13, the control sample is compared with a primary sample chlorinated at the Oak Ridge West Wastewater Treatment Plant with a 10 mg/l chlorine dose to a chlorine residual of 0.9 mg/l. Some chromatographic peaks are apparently destroyed by chlorination and other peaks are formed during disinfection with chlorine. As previously reported, some of these constituents formed during chlorination are probably chloro-organics (3, 7, 8, 10, 12, 13). In Figure 8, the control sample is compared with the primary sample after ozonation with 6.9mg ozone/liter effluent. The high-oxidation potential of ozone is clearly evident in the destruction of most of the chromatographic peaks.

In Figure 14, the control sample is compared with the primary sample after disinfection with 27,500

TABLE 5. COMPARATIVE DISINFECTION OF PRIMARY WASTEWATER EFFLUENT SAMPLE: CHEMICAL AND DISINFECTION DATA.

Sample	pH	BOD	COD	TOC	TKN	Total [a] coliform (No./100 ml)
		mg/liter				
Control	7.6	14	28	20.1	9.2	185,000
Chlorinated (Cl residual, 0.9 mg/liter)	7.7	19	42	20.7	8.6	<1
Ozonated (6.9 mg ozone/liter)	7.9	12	32	17.6	8.4	15
Ultraviolet irradiated (27,500μ W•sec/cm^2)	7.6	18	42	17.9	8.8	<1

(a) Membrane-filter method (2).

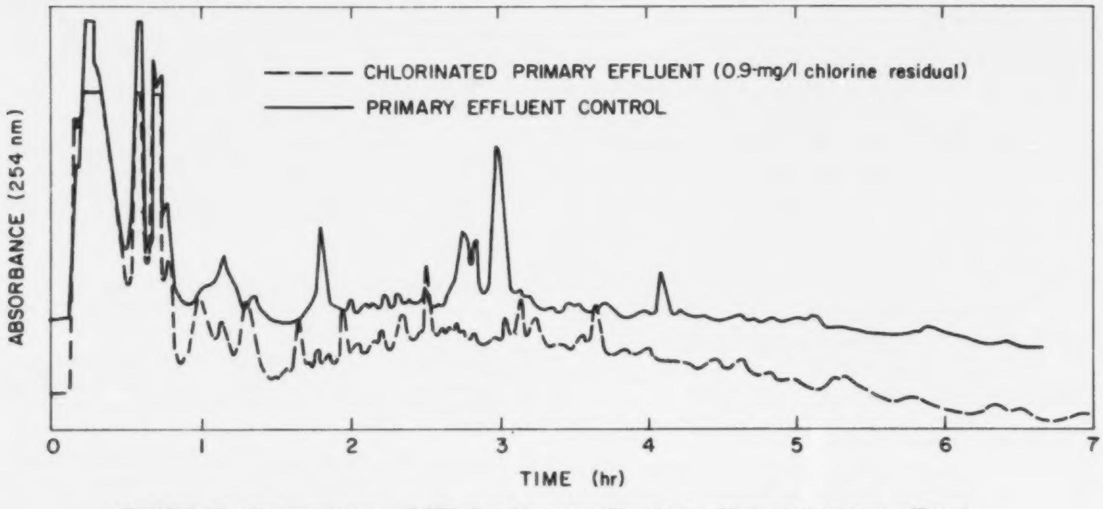

FIGURE 13. Chromatograms of UV-absorbing constituents in chlorinated primary effluent (chlorine residual, 0.9 ppm) compared with undisinfected control primary effluent. Samples were collected from the Oak Ridge West Wastewater Treatment Plant. Chlorinated sample, 0.77 ml of 800X concentrate. Unchlorinated sample, 0.77 ml of 700X concentrate.

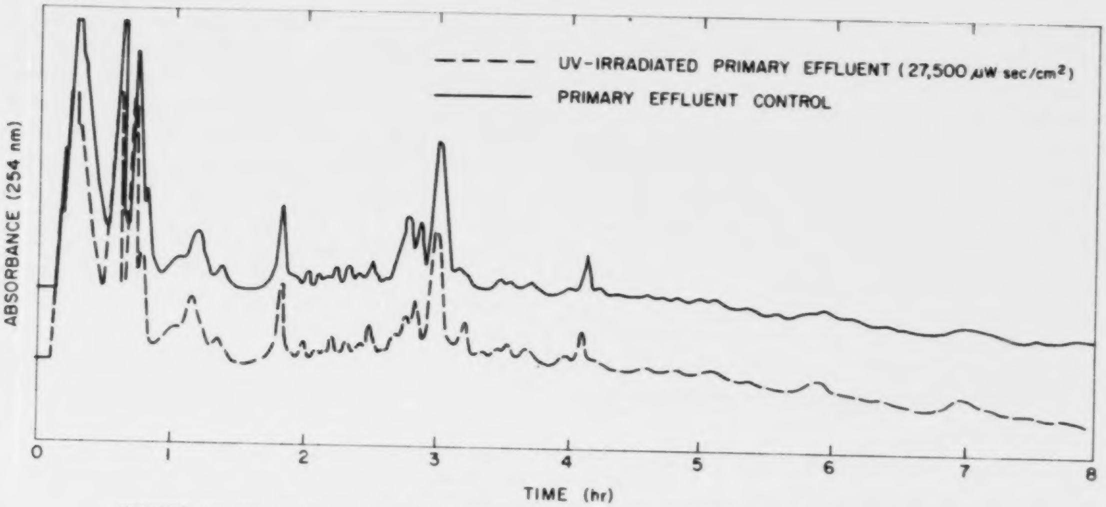

FIGURE 14. Chromatograms of UV-absorbing constituents in UV irradiated primary effluent (27,500 W•sec/cm²) compared with undisinfected control primary effluent. Samples were collected from the Oak Ridge West Wastewater Treatment Plant, and the disinfected sample was UV irradiated in the laboratory. Ultraviolet irradiated sample, 0.77 ml of 660X concentrate. Unirradiated sample, 0.77 ml of 700X concentrate.

μW•sec/cm² of UV irradiation. The apparent absence of chemical effect on UV-absorbing constituents is clearly evident from the close agreement between the two chromatograms. This lack of chemical effect by UV irradiation at levels that disinfect the wastewater may be highly significant and is being investigated further.

CONCLUSIONS

Disinfection of wastewater effluents by either chlorination or ozonation destroys many nonvolatile organic constituents and produces other nonvolatile compounds. Disinfection of wastewater effluents by UV-light irradiation has relatively little effect on nonvolatile chromatographic constituents. Methods have been developed for preparing effluent concentrates and nonvolatile chromatographic constituents for testing for mutagenic activity. The next phase of this research program will emphasize characterization and identification of compounds having mutagenic activity.

ACKNOWLEDGEMENTS

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25.

MUTAGENIC ACTIVITY OF NONVOLATILE ORGANICS DERIVED FROM TREATED AND UNTREATED WASTEWATER EFFLUENTS

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INTRODUCTION

Wastewater disinfection is an important element in the continuing battle to control infectious disease. Adequate disinfection becomes more critical as population size increases and as people live closer together. Thus, on a global scale the effectiveness of water disinfection is a matter of vital concern.

Disinfection technologies, like other large-scale technologies, have routine and potential costs as well as benefits associated with them. It is now well established (4) that a large number of chlorine containing organics are produced during the disinfection of wastewaters by chlorination. Other disinfection processes also have the potential for producing modified organics, but most of the compounds so produced have yet to be identified. Some of the nonvolatile chlorine containing organics produced in wastewater by chlorination are relatively stable and are known to show great biological activity. If released in sufficient quantity, they have the potential for damaging the environment and/or causing adverse effects on human health.

Disinfection processes which have been proposed as alternatives to chlorination also have the potential for releasing modified organics of unknown biological activity to the environment. That ozonation modifies the spectrum of nonvolatile organics in wastewater effluents has been clearly demonstrated (5), but for other alternative disinfection procedures the situation is less clear. The identity of biological consequences of modified organics produced by these alternative technologies are entirely unknown.

Mutagenicity is one biological endpoint which is relevant to safety related evaluation of environmental chemicals. A significant increase in the mutation rate could be a direct chronic health hazard to humans or could adversely affect animal or plant populations and thus profoundly damage the environment. In addition short-term tests for mutagenicity have been highly correlated with mammalian carcinogenicity (1) and thus may give a preliminary indication of danger of an increase in human cancer.

We have performed several biological tests on treated and untreated wastewater effluents. The present paper is limited to tests for mutagenicity using several indicator strains of enteric bacteria. While this biological screening program is in its early stages, the results to date already indicate that the problem of mutagenicity of modified organics in disinfected wastewater effluents is one that needs to be taken very seriously in planning the disinfection strategy for the future.

MATERIALS AND METHODS

Bacterial Mutagenicity Testing

The data reported here were obtained using standard bacterial tester strains and protocols. *Escherichia coli* strain WP2 was used according to procedures outlined by Green and Muriel (3). This strain measures reversion to tryptophan prototrophy by a base-pair substitution mechanism and was used in a suspension test in which the bacteria were exposed to the presumptive mutagen for different periods of time and then plated for survival and mutation frequency. We also used four tester strains of *Salmonella typhimurium* developed by Dr. B. N. Ames and his collaborators which test for reversion to histidine prototrophy. The strains used were TA-1535 and TA-100 which are sensitive to base-pair substitution mutagens, and TA-1538 and TA-98 which detect mutagens which operate by a frame-shift mechanism. These strains were used in a plate test in which the bacteria are included with the presumptive mutagen in a top agar overlay, and the effect is scored as mutant colonies per plate. The procedures used are detailed by Ames *et al.* (2).

Sample Collection, Handling, and Concentration

Samples of wastewater effluents were collected, quick frozen, and shipped to the Oak Ridge National laboratory on solid CO_2. Once in the laboratory the samples are concentrated 1000-3000 fold by lyophilization and subsequent rehydration in a suitable volume of distilled water. The details of the sample handling and concentration procedures are presented elsewhere (5).

Sample Sources

The samples were obtained from several wastewater treatment plants and sampling was always done before and after the final disinfection step. The samples were obtained from the plants listed in Table 1.

RESULTS

Preliminary Experiments

The testing of complex mixtures for mutagenic potential presents special problems and any separation technique currently available carries with it the possibility for the loss or inactivation of mutagenic components. The most sensitive or useful tester strains for nonvolatile organic materials cannot be determined from negative results. We have, therefore, tested standard Oak Ridge primary and secondary effluent concentrates extensively in a series of preliminary experiments using a variety of bacterial tester strains in an effort to optimize the conditions for the detection of mutagens among the nonvolatile organics of wastewater effluents.

TABLE 1. SOURCES OF SAMPLES TESTED FOR MUTAGENIC ACTIVITY

Designation	Name of Treatment Plant	Location	Type of Treatment	Disinfection Method	Type of Wastewater
ORPE	Oak Ridge West End Wastewater Treatment Plant	Oak Ridge, TN	Primary	Chlorine	Primarily residential
ORSE	Oak Ridge East End Wastewater Treatment Plant	Oak Ridge, TN	Secondary	Chlorine	Primarily residential
CSE	Taft Center USEPA Pilot Wastewater Treatment Plant	Cincinnati, OH	Secondary	Ozone	Residential and Industrial
EPSE	Upper Thompson Sanitation District	Estes Park, CO	Secondary	Ozone	Residential, relatively clean
NBCSE	Northwest Bergen County Wastewater Treatment Plant	Waldwick, NJ	Secondary	UV	Primarily residential
MCSE	Meander Creek Wastewater Treatment Plant	Mahoning County, OH	Secondary	Ozone	Primarily residential
LBSE	Lyons Bend Wastewater Treatment Plant	Knoxville, TN	Secondary	Chlorine	Primarily residential

In these preliminary experiments we did obtain positive results with the *Salmonella* strain TA-1535 in some samples. TA-100 gave only weak response to samples which were positive for TA-1535, but we retained it in many subsequent tests because for some types of samples it is similar to, but more sensitive than, TA-1535. The other Ames tester strains, TA-1538 and TA-98, which detect primarily frameshift mutagens, showed no response in our preliminary experiments and we used them rarely in subsequent tests. The *E. coli* strain WP2 which, like TA-1535, should detect base-pair substitution mutations, also showed no positive response in our preliminary experiments. We did use WP2 in many experiments because in our hands it had previously given results similar to TA-1535 for other kinds of samples.

Testing of Field Samples

Table 2 summarizes results on field samples from 7

High Pressure Liquid Chromatography

A sample which proved to be mutagenic as a crude concentrate was subjected to HPLC as described elsewhere (5). Fractions were collected and pooled to make subsamples. In one case the 220 fractions were pooled to give 29 samples, most of which represented individual UV absorbing peaks on the chromatogram (Table 3, Figure 2). In the other case the 200 samples were pooled to give four samples (Table 3, Figure 3). These samples were lyophilized, rehydrated and tested for mutagenic activity with strain TA-1535.

Presentation of Data

For the purposes of this report tests which showed 2 times the activity of contemporary negative controls or less are presented as "negative" (−). Those which showed from 2 to 5 times the activity of the negative controls are called "weakly positive" (±). Those which showed 5 or more times the activity of negative controls are considered positive (+).

wastewater treatment plants. Most of the tests show negative results. We obtained no evidence for mutagenicity for either chlorinated or unchlorinated samples from the Oak Ridge east end wastewater treatment plant (ORSE) using a variety of tester strains. This plant treats primarily residential wastewater and carries out secondary treatment. In contrast, we obtained clear positive results for chlorinated samples from the Oak Ridge West End Wastewater Treatment Plant (ORPE) in strain TA-1535. There was no positive effect for these samples in WP2 and only a slight effect in TA-100. The positive effect of these samples is substantial (see Figure 1), and it was repeated many times in samples taken at different times of the year. The positive mutagenic effects in samples from ORPE is dependent upon chlorination — unchlorinated samples showed no mutagenicity. The Oak Ridge west end plant is a primary treatment plant and this is the only primary treatment plant included in this study (see Table 1).

A weak positive effect was seen with or without chlorination in effluents from the Lyons Bend wastewater treatment plant from Knoxville, Tennessee. This plant serves a primarily residential area. The effect was seen only with TA-1535. Clearly more testing needs to be done on Lyons Bend. To date, we have looked at only a single sample from this plant.

The other plants tested (CSE, EPSE, NBCSE, and MCSE) have to date given only negative results in the strains used. This does not mean that there are no mutagenic components in these treated or untreated effluents, but only that with the strains and procedures we have used to date, such components have not been detected.

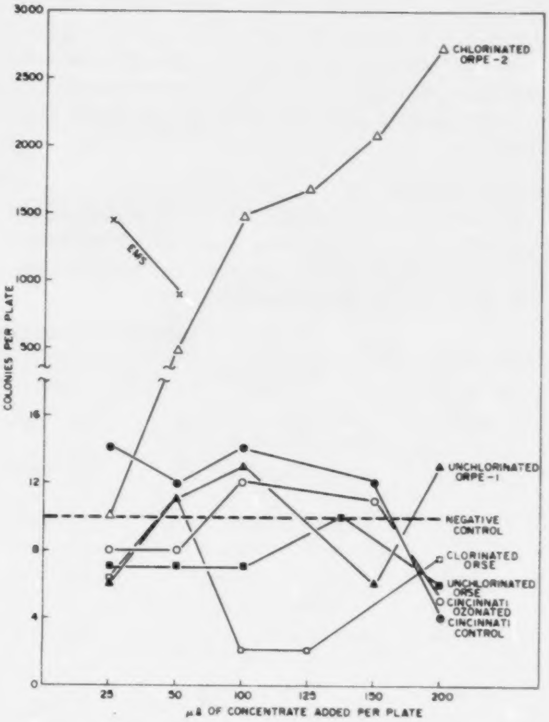

Figure 1. Mutagenic activity tests of effluent concentrates using *Salmonella typhimurium* strain TA-1535.

TABLE 2. RESULTS OF MUTAGENICITY TESTS ON SOME WASTEWATER CONCENTRATES

Sample* Source	Disinfection Treatment	Treatment Dose	Sample Date	Concentration Factor	Tester Strain	Mutagenic Activity
ORSE	none	—	2/7/77	1000	WP2	-
ORSE	none	—	9/27/77	1000	WP2	-
ORSE	none	—	10/5/77	2350	TA-1535 TA-100	- -
ORSE	none	—	2/8/78	2000	WP2	-
ORSE	chlorine	0.5 ppm residual	8/2/77	3100	WP2	-
ORSE	chlorine	0.2 ppm residual	9/27/77	1000	WP2	-
ORSE	chlorine	0.5 ppm residual	10/5/77	2200	TA-1535 TA-100	- -
ORSE	chlorine	0.8 ppm residual	1/31/78	2000	WP2	-
ORSE	ozone	8.0 mg/l	10/5/77	2200	WP2 TA-1535	- -
ORSE	U.V.	21,208 μWsec/cm²	10/5/77	2350	WP2 TA-1535	- -
ORPE	none	—	9/27/77	1000	WP2 TA-1535	- -

TABLE 2. RESULTS OF MUTAGENICITY TESTS ON SOME WASTEWATER CONCENTRATES (Continued)

Sample* Source	Disinfection Treatment	Treatment Dose	Sample Date	Concentration Factor	Tester Strain	Mutagenic Activity
ORPE	none	—	12/12/77	2000	TA-1535	-
ORPE	none	—	5/19/78	2000	TA-1535	-
ORPE	chlorine	1.0 ppm residual	9/27/77	1000	WP2	-
					TA-1535	+
ORPE	chlorine	0.9 ppm residual	12/12/77	2000	WP2	-
					TA-1535	+
					TA-100	±
ORPE	chlorine	1.0 ppm residual	5/19/78	2000	TA-1535	+
ORPE	ozone	6.9 mg/l	12/12/77	2000	WP2	-
					TA-1535	-
					TA-100	-
ORPE	U.V.	27,500 μWsec/cm^2	12/12/77	2000	TA-1535	-
CSE	none	—	9/12/77	1500	WP2	-
					TA-1535	-
					TA-100	-
CSE	ozone	8.4 mg/l	9/12/77	1600	WP2	-
					TA-1535	-
					TA-100	-
CSE	none	—	5/29/78	2330	TA-1535	-
					TA-100	-
CSE	ozone	8.2 mg/l	5/24/78	2150	TA-1535	-
					TA-100	-
ESPE	none	—	10/19/78	2000	WP2	-
					TA-1535	-
					TA-100	-
ESPE	ozone	~6 mg/l	10/19/78	2000	WP2	-
					TA-1535	-
					TA-100	-
NBCSE	none	—	4/21/78	3330	WP2	-
					TA-1535	-
					TA-100	-
NBCSE	U.V.	35,000 μWsec/cm^2	4/21/78	2700	WP2	-
					TA-1535	-
					TA-100	-
MCSE	none	—	6/15/78	2000	TA-1535	-
					TA-100	-
					TA-1538	-
					TA-98	-
MCSE	ozone	~6 mg/l	6/15/78	2000	TA-1535	-
					TA-100	-
					TA-1538	-
					TA-98	-
LBSE	none	—	5/3/78	2000	WP2	-
					TA-1535	±
					TA-100	-
LBSE	chlorine	0.9 ppm residual	5/3/78	1667	WP2	-
					TA-1535	±
					TA-100	-

*See Table 1.

Testing of Fractions Separated by High Pressure Liquid Chromatography

In an attempt to identify the mutagenic compound(s) in ORPE chlorinated wastewater responsible for the strongly positive response in TA-1535, we subjected one such mutagenic concentrate to HPLC. Samples were collected and pooled to give subsamples of the original concentrate (see Materials and Methods). Of the 29 samples so obtained from one such fractionation, eight were weakly positive in a mutational assay with TA-1535 (see Table 3 and Figure 2). In another HPLC separation the fractions were pooled to obtain four samples and these were tested for mutagenicity with TA-1535. Samples 2 and 3 were positive (see Table 3 and Figure 3). In both cases the total mutagenicity recovered was far less than present in the original concentrate.

TABLE 3. RESULTS OF TESTS OF CHROMATOGRAPHIC FRACTIONS OF CHLORINATED PRIMARY EFFLUENT FROM OAK RIDGE WASTEWATER TREATMENT PLANT (MAY 19, 1978).

Chromatographic Separation No. 1 (Figure 2)					Chromatographic Separation No. 2 (Figure 3)			
Sample Number	Fraction Number	Peak (UV) Present	Result TA 1535	Number of Tests	Cut Number	Fraction Number	Result TA 1535	Number of Tests
1	1-7	+	−	1	1	1-62	Negative	2
2	8-10	+	+	2				
3	11-15	+	+	2				
4	16-20	+	−	1				
5	21-23	+	−	1				
6	24-28	+	−	1				
7	29-32	+	−	1				
8	33-37	+	+	2				
9	38-43	+	−	1				
10	44-47	+	−	1				
11	48-51	+	+	2				
12	52-58	+	−	1				
13	59-62	−	−	1				
14	63-68	−	−	1	2	64-124	Positive	2
15	69-73	+	−	1				
16	74-77	−	−	1				
17	78-80	+	−	1				
18	81-88	+	+	2				
19	89-94	+	−	1				
20	95-107	+	−	1				
21	108-114	+	−	1				
22	115-124	−	−	1				
23	125-133	+	−	1	3	125-186	Positive	2
24	134-145	+	−	1				
25	146-155	+	+	2				
26	156-170	−	+	2				
27	171-185	−	−	1				
28	186-200	+	+	2	4	187-200	Negative	2
29	201-220	−	−	1				
Total				36				8

Tests run with appropriate positive and negative controls.

DISCUSSION

The problem of measuring mutagenicity in treated and untreated wastewater effluents is very complex. Such effluents contain a large number of organic compounds, some of which are quite toxic to the test systems currently available. In addition, there is tremendous variation in the content of effluents from one treatment plant to another, making generalizations very difficult. At present, studies must proceed on a plant by plant basis. The primary goal of this type of work is to evolve general principles for predicting the production of mutagenic and carcinogenic materials during the disinfection process, and to achieve an understanding of the conditions under which such materials are likely to be produced. This would lead to the technology necessary for adequate disinfection without increasing the burden of chronic disease in the human population.

Mutagenicity of some chlorinated wastewater effluents has been demonstrated, and in spite of imperfections in the testing technology, the reality of the effect is not in doubt. The data obtained in the HPLC separations suggest the possibility of complex interactions between various components of the concentrate. Areas of the chromatogram which are weakly mutagenic in the first experiment show no effect when tested in larger groupings in the second experiment (compare Figures 2 and 3). The total mutagenicity of the fractions from HPLC when tested separately is far less than the mutagenicity seen in the original crude mixture. This suggests that some of the mutagenic agents present in the crude concentrates are either lost or inactivated in passage through this type of column. This is an important, but not unexpected, finding with implications for the planning of further studies. It is important to identify specific mutagenic compounds in treated wastewater effluents, but if they are lost or inactivated by the best available separation techniques, the job becomes extremely difficult.

The data presented here, though preliminary, indicate a potentially serious problem which must be considered in the planning of the disinfection needs for society in the future. Therefore, high priority should be given to further research in this area. It is important to produce the data which are necessary to make a complete evaluation of the problem, and given the complexity of the scientific questions which have been raised, this will require a considerable period of time.

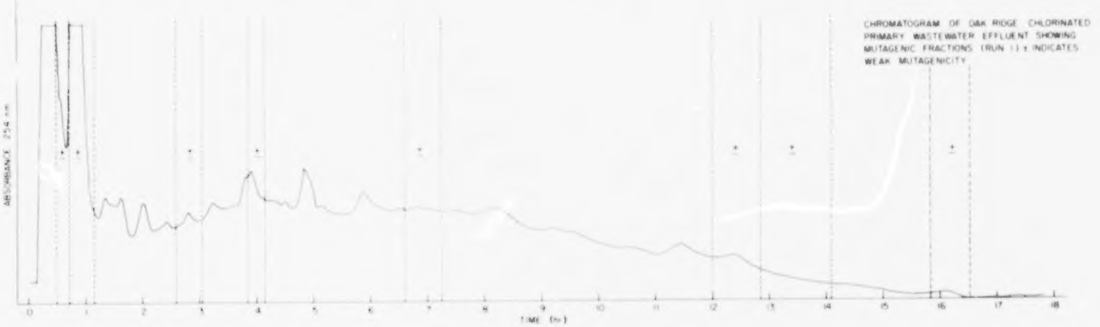

Figure 2. Chromatogram of Oak Ridge chlorinated primary effluent showing mutagenic fractions (Run 1). ±indicates weak mutagenicity.

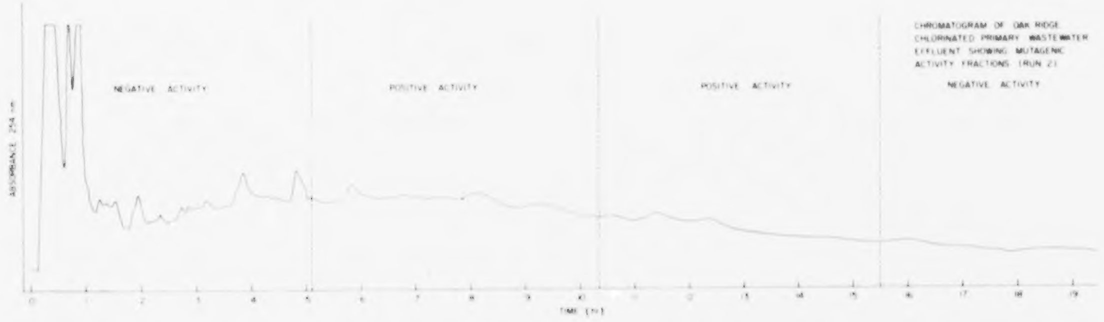

Figure 3. Chromatogram of Oak Ridge chlorinated primary effluent showing mutagenic fractions (Run 2).

Estimating the potential for increased risk to adverse human health effects is the ultimate goal of research of the type described here. This implies a quantitative estimate which could be used in balancing the risks with the benefits for the various disinfection technologies. However, we are now at the stage of trying to gain insights into potential risks on a purely qualitative level. This is all that the current state of the scientific art will allow. We are still a long way from being able to quantify the problems which are becoming all too apparent on a purely descriptive level.

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26.
PANEL DISCUSSION OF VIRUS, NON-VOLATILE ORGANICS, AND MUTAGENIC EFFECTS

Participants in Viruses and Organics Roundtable Discussion

Edward J. Opatken, Moderator
U.S. EPA-Cincinnati

1. Robert Fluegge
   Carborundum Company
2. Robert Jolley
   Oak Ridge National Laboratory
3. Robert Cumming
   Oak Ridge National Laboratory

DR. LONGLEY: I have a comment first of all. In the last break I talked to Al Venosa and apparently I did not define myself fully enough when I had talked earlier. We have today a situation where the EPA and other groups, ASTM as an example, are putting out standards. There is no coordination that is very effective between these groups. On the biological side, I have to give a lot of credit to Ed Geldreich for the efforts that he has made to make the EPA standard methods have some agreement, but in general, in residual analysis we have no strong central guilding light to put us all down the same path and this is creating a lot of problems.

This, I believe and a number of other people I have talked to believe, is something that we have to address soon. It is a problem and if we do not establish coordination, it is just simply going to hurt the field of disinfection in years to come.

Are we open for questions to the panel now?

MR. OPATKEN: Yes, we are.

DR. LONGLEY: Thank you. Mr. Fluegge, I have two questions to ask you on the work that you presented. In particular, you mentioned that you are using pre-filters. I understand from what you said that the pre-filters were not used in all instances, is that correct?

MR. FLUEGGE: That is correct. Whenever the total suspended solids were sufficiently high that the on-site field engineer due to his experience would determine that the flow rate would drop too far prior to collecting the full 100 gallons, then we would add the clarifier filter in front of the addition of all chemicals.

DR. LONGLEY: When you did that, did you then attempt to elute virus in the solids that you picked up on the pre-filter?

MR. FLUEGGE: We did in each case. We did find virus in the clarifiers as well as in the adsorber, as I mentioned in the presentation. The number of viruses collected on the clarifier are usually smaller than the number of viruses that are picked up on the adsorbing filter. This is not always the case but most of the time.

DR. LONGLEY: In San Antonio at the Rilling Road plant, which is a 90 mgd plant, we saw the same phenomenon that you saw. One possible mechanism leading to that is the fact that the waters which are entering that plant prior to that large peak are lower in organic and other materials which would be adsorbed on the floc, in the case of the San Antonio plant which is an extended aeration plant. We are hypothesizing that one mechanism might be that we are simply beginning to elute some of the virus off of the floc that have been adsorbed on the floc in the plant and we are seeing them in the effluent. I do not know if that explanation would be applicable to what you saw . . .

MR. FLUEGGE: You are thinking that the floc ends up on the clarifier . . .

DR. LONGLEY: I am saying that the floc in the plant has adsorbed virus and, as the load coming into that chamber is lower in organics and other materials which will adsorb to that floc, there is less competition for sites. We are beginning to elute for a period of time some of the virus in the plant off of the standard aeration.

MR. FLUEGGE: Usually the pH has to change a quite drastic amount.

DR. LONGLEY: That is another factor, too.

MR. VENOSA: Karl, just a quick question on your San Antonio effluent. Did you find relatively the same number of viruses as Mr. Fluegge found?

DR. LONGLEY: I was not looking at the numbers so closely on his graph. I just saw his peak. You were isolating animal virus. We were looking at phage, so the numbers would not necessarily be comparable.

Just one other comment. Your animal virus numbers are much lower, if I am not mistaken, than the viruses reported sometime ago in Gerry Berg's book. Am I correct on that?

MR. FLUEGGE: I believe that is correct, yes.

DR. LONGLEY: They are like maybe three orders of magnitude in fact.

DR. JOHNSON: Dr. Cumming, your results are really impressive. The question is, what kinds of compounds does this particular *Salmonella* mutagenesis test respond to? I know that there has been a comment made about several of the Ames *Salmonella* strains not responding very well to chlorinated compounds. That is, you could have a chlorinated compound which would be mutagenic in animals but not sensitive in the *Salmonella* test, which is I guess the standard test. What kinds of compounds would be expected to provide the kind of transformations that you see in your testing?

DR. CUMMING: Well, the strain TA-1535 presumably detects base pair substitutions mutations. The Ames base pair substitution strains are missense mutants and the precise molecular basis is not as well established as it is for some of the *E. coli* strains.

TA-100 has the same mutation as TA-1535 but has an addition of an episome (an R factor), which makes it more sensitive to most compounds and very much less sensitive to what we are seeing in the sewage. That is very similar to a result observed by others with some relatively small chlorinated compounds. That is, TA-1535 being more sensitive than TA-100. Specifically, certain nitrosamines, which are not mutagenic without metabolic activation, become mutagenic if they are chlorinated. Those compounds respond more strongly to TA-1534 than they do to TA-100. So, whatever we have is like that. That does not give any indication as to what the nature of the beast is, but this does establish a precedent for certain chlorinated compounds being more strongly effective in the particular strain we did see the activity in.

DR. JOHNSON: Can you explain base pair substitution a little bit? In other words, are you talking about the nucleic acid natural bases being substituted, like the classic fluoro-uracil type compound?

DR. CUMMING: Right. Incidentally, since you mentioned it, we have looked at chloro-uracil and see nothing in any of the Ames strains with it, but we do see a positive effect in *E. coli.*

Base pair substitutions are the change in the genetic code by substituting one nucleic acid base for a wrong one. It's a mis-substitution, and in general I do not want to get too technical on the mutagenic theory, but there are two types.

One, so called missense mutation, simply inserts a wrong amino acid in a protein, so you have a defective protein as the end result. The other, called nonsense mutation, inserts a signal which terminates the protein, and the *E. coli* strain we use is of that type. The Ames strains are missense; they just give a defective protein.

DR. JOHNSON: Very exciting!

MR. DeSTEFANO: I have a question for Dr. Cumming. I brought this up yesterday and had I known you would be here, I think I would have reseerved the question. Do you think it would be possible that, with ultraviolet light or some other form of disinfection, you could inhibit the ability of *E. coli* to ferment lactose and therefore not appear on the MPN test?

DR. CUMMING: Yes, I presume it is possible. Certainly one thing that we have to be concerned about is that when you disinfect you have a surviving fraction, and the surviving fraction may be different biologically from what you started out with. It can be different for a lot of reasons but mutagenesis is certainly a factor.

MR. BLAINE SEVERIN: I have a question for Mr. Fluegge. Are we to understand from your information that none of the three disinfectants are really killing the virus present and that we cannot claim that one is better than the other? I have heard claims that ozone kills more virus than chlorine and that UV will kill more virus than chlorine and in this real situation disregarding the question about solids elutriation and the primary filter, is there a difference? Is one better than the other?

MR. FLUEGGE: All three disinfectants ended up with virus in the effluent to the stream. There was not one other than chlorine at one plant which had a low input and we did not pick up any in the output, but we did find an output in a chlorine disinfection scheme at Muddy Creek. At the beginning of the program we tried to design the experiment to look at one versus the other and it is not clear to me that we could pick one kind of disinfectant as being better than the other, especially if you compared ozone and chlorine at Muddy Creek. UV looked good at NW Bergen County. On the other hand, I had no comparison with chlorine as a base or with ozone.

DR. BILL CHAPPELL, *University of Colorado*: We are doing some work that is similar to the Oak Ridge work in many ways but in the context of potable reuse of wastewater, which as some of you may know Denver is interested in. We are looking at the output from the Denver metro plant. The chemistry is being carried out by Harold Walton of the Chemistry Department at CU and the Ames testing by Dr. Lee Spalbender of the American Medical Center. I would like to relate a few of our own results. We are in a much earlier stage, but we have seen strong mutagenic activity in the chlorinated effluent from the metro sewage plant. It has involved both base substitution and frame shift mutations, but there seems to be a strong time variation because from one time to the next we will see very strong mutagenic activity and then another time, essentially no activity at all.

MR. VENOSA: Bill, is the metro sewage plant a secondary or a primary plant?

DR. CHAPPELL: Secondary.

MR. VENOSA: I think it should be emphasized here that the study Bob Cumming reported positive results on was on primary effluent.

DR. CUMMING: The positive effects were from primaries and they were dependent upon chlorination. I do not know whether your results were also dependent upon chlorination.

DR. CHAPPELL: Yes, they were.

MR. WARRINER: A couple of methodology questions for the Carborundum procedure. I gather you passaged 50% of your isolations and, if that is correct, I would like to know what fraction of that 50% was confirmed for anything?

MR. FLUEGGE: All of the plaques were passed from one cell culture to another. The limitation of the LBM serum pool was part of the reason for not identifying all of them as well as the time associated with that.

MR. WARRINER: Did you find false plaques in the first time around?

MR. FLUEGGE: False plaques? Yes.

MR. WARRINER: What percentage?

MR. FLUEGGE: I could not say. Dr. Metcalf would be more able to give a percentage. I do not end up with that particular percent number in Dr. Metcalf's reports. I get the positives coming through.

MR. WARRINER: Could I ask if you do calibration runs using one or another of the viruses you identified or something related to them?

MR. FLUEGGE: We did a number of seed runs, as people will call them, putting poliovirus 1 seed into waters and then recovering them. There are a lot of difficulties in trying to understand what a seed run truly means.

When you put high titers of virus, like 10^6 per ml or 10^7 per ml into 50 gallons and then start concentrating through the filters, elute, and analyze and determine how many viruses are picked up after you've pulled them out, that takes care of the numerator quite well. You have to worry about what is in the denominator. How many did you really put into that water? Now, you have 1 ml of liquid and you are putting it into this 50 gallons. You are going to have to mix that 50 gallons. We try very hard to get a mono-dispensed set of viruses throughout that 50 gallons. The question is: are there other chemicals, are there materials within that 50 gallons that can kill those viruses you just put in? If you just simply said, I had 10^6 per ml and now I have this many that I got back, then you can end up with a very conservative percentage recovery. With that kind of calculation, we get numbers coming out somewhere between 5 and 30% of the number put in.

Then, it can go the other way. If you put it into, say, 5 gallons, and you let that seed sit around for the 2 to 3 hours that you had spent collecting your sample, and then you take a grab sample out, you could get 130% recoveries, which would indicate to me that we had better start running. So, it is very difficult to get that denominator, large or small. If it is distilled water or distilled water with a little salt solution in it, you are in good shape. It is when you get out into the real world, where you may have all kinds of chemicals that may cause your virus to clump together or cause them to die or whatever, where the real rub comes.

MR. WARRINER: You say you do not have a recovery factor we can put on the numbers that you had in your graph.

MR. FLUEGGE: One of the reasons for designing the experiment the way we did, namely, before and after the adding of disinfectant, was that whatever was in the water during that 2 hour period, was true for both cases, those that were not having the disinfectant added and those that did have it added. Thus, we could compare the two. Even though you cannot go to an absolute basis, you can compare the number of viruses that entered say the UV light versus the number of viruses that left the UV light sampling the chlorine.

MR. ELLNER, *Ultraviolet Purification Systems*: I would like to first point out some very serious inadequacies in the reportage here. In one of the first discussions, the ultraviolet source was described as a 100 watt lamp with 30% of its output in the ultraviolet range. Now that would be akin to somebody talking about visible light and saying I used 100 watt lightbulb with 70% of its output in visible light. You could have 100 watt fluorescent lightbulb one inch long with an extremely high light intensity per inch of length. You can have 100 watt neon light tube. You could have 100 watt quartz iodine light tube and the point is that in describing ultraviolet lights the method of describing the output would be in microwatts per square centimeter at a given distance, because there is no way that anybody reviewing this information can have any idea as to the actual output per unit length of the lamp. Also, the length of the lamp is not given.

We see variations with what is apparently the same equipment. One reporter claims 180,000 microwatt seconds per square centimeter while another reporter talks about 21,000 microwatt seconds per square centimeter. There is no information regarding the ultraviolet transmission, as measured in a spectrophotometer, of the liquid being treated, which can have a complete and total effect on the results. If one of the effluents had a 90% transmission and the other effluent had a 10% transmission, this is going to have a tremendously important and meaningful effect on what the results are. If we are trying to discuss quantitative results here, I suggest that before items are published, a little more serious thought be given to the descriptions and the use of quantitative terms, primarily dosage. Especially, the dosage information is totally meaningless unless you relate it to the transmission. The dosage capability of the system is directly related to the transmission of the particular liquid. You can take that device and put a material with zero color and zero turbidity, let's call it kerosene for this example . . .

MR. OPATKEN: Sid, this session is devoted to the questions in regard to the organics and the viruses and so forth . . .

MR. ELLNER: Well, there were statements made regarding virus and the people here are obviously going to come to conclusions regarding the ability of ultraviolet, not a given wavelength, but of the technique to either handle or not handle virus. Unless you have meaningful . . .

MR. OPATKEN: True, but if you have a question, you can address it to the board.

MR. ELLNER: My question is this. How did you arrive at your dose? What was the ultraviolet transmission? What were the flow rates and what were the lengths of the lamp?

MR. FLUEGGE: Well, first of all the plant was described yesterday by Karl Scheible of Hydroscience in quite a bit more detail than I tried to cover today. Regarding the number that I calculated, I just simply took the wattage output at 30 watts for each bulb. I took into account the length of the bulb as being five feet and the diameter of the bulb as being 0.9 inches. I then calculated the number of photons leaving that bulb a quarter of an inch away. Now, what happens is that only about 120 of the 400 lamps take up the outer part of this whole array. That means that there are 280 bulbs that are all inside, and so all of those bulbs are simply radiating to other lightbulbs. That photon ends up going into another lightbulb and then re-emitting. I had no way of determining how many photons are bouncing around inside of this thing. I do not see how anybody can really get a handle on how many photons the viruses are being exposed to without some sort of chemical actinometry or chemical calibration. I did not have any way of doing that in the output of a regular plant.

MR. ELLNER: And that is why I am saying you should not report what the dosage was then.

DR. JOLLEY: Mr. Chairman, before you adjourn, I would like to make a comment, too. I think I would like to focus back again on the significance of the mutagenic activity in the chlorinated primary effluent. It was somewhat dismissed by comment that it is just primary.

There are many major cities which currently discharge their primary into waterways. For example, St. Louis discharges a considerable amount.

Furthermore, the quality of the primary sewage that we are dealing with in Oak Ridge, for example, is equivalent to many secondaries. It is a very clean, clean sewage. The fundamental question to me is whether or not indeed there is something in activated sludge that does reduce this mutagenic activity, or whether or not it is a concentration phenomenon and we are just not seeing it and it is still there.

Then I would like to make a statement relative to THM's which I missed. The question came up yesterday or the day before concerning production of THM's in these disinfection processes.

We measured THM's with respect to our UV irradiation and found essentially no difference between the influent and the effluent. With respect to ozonation at neutral pH we found little difference. Some would say that ozone would convert the chloride into chlorine because of its potential. This is a very slow reaction. Furthermore, there is a lot of ammonia there. Now whether or not there would be any THM's formed in a system without ammonia I do not know.

With respect to chlorination of sewage, the same concentrations of THM in influent and effluent were measured in effluents that had ammonia above about 1 ppm. In nitrified sewage there was considerable amount of chloroform produced on chlorination, and so there is that potential when you remove the ammonia to produce THM's on chlorination.

MR. DONALD BERMAN: For Mr. Fluegge. I am curious . . . was there any die-off during transport of the sample from the field to the laboratory?

MR. FLUEGGE: We have looked at this on other studies, not in this particular study, but we have looked at survival of virus at dry ice temperatures. The concentrate is put into a vial and put at −78° C. At that temperature the die-off has been very low in our previous studies. I cannot answer the question in this one because I could not assay in the field to determine the difference.

MR. BERMAN: The second question to go along with this. Did you have any trouble at all getting samples back to the laboratory? I am curious as to how you shipped it, because this a potential etiologic agent. Did you just have it frozen and then ship it without any special warning label or what have you?

MR. FLUEGGE: We label each of our samples as they should be labeled. We follow the Public Health Laws, the Federal Laws and the carrier law requirements as we ship. It is not easy. I does take a lot of paperwork. You do have to fill out all kinds of forms. You have to label it correctly and your biggest problem is when you conduct a seed experiment. How do you get that to the field and how do you get it back home. There is where your concentrations are known in your sample. You put them there.

SECTION 7. PLANNING AND IMPLEMENTATION

27.

PLANNING DECISIONS IN SELECTING WASTEWATER EFFLUENT DISINFECTION FOR THE OLENTANGY ENVIRONMENTAL CONTROL CENTER

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ABSTRACT

Region V of the U. S. Environmental Protection Agency prepared an Environmental Impact Statement (EIS) for the Olentangy Environmental Control Center in Delaware County, Ohio. The new interceptors and treatment plant would provide tertiary treatment for 5,700 m^3/d (1.5 mgd) and later 11,000 m^3/d (3 mgd) of wastewater. Discharge will be to a segment of the Olentangy River noteworthy for its biological quality and natural character. Mitigation was necessary to protect the biota from potential adverse chlorine impacts at low flow. Ozone was selected as the most cost-effective disinfection alternative. The EIS was a successful tool for selecting a new technology for wastewater disinfection.

INTRODUCTION AND OBJECTIVES

The National Environmental Policy Act of 1969, known as NEPA (1), established the Environmental Impact Statement, or EIS, process. Much has been stated in documents, books, and legal decisions about what an EIS should be and what it should do (2). The fine points of this remain a continuing debate, as shown in the proposed Council of Environmental Quality's (CEQ) regulations, which apply to the EIS's of all Federal agencies (3). At its best, we believe that the EIS process gives the Federal agency the opportunity to examine the environmental implications of its actions, and expands the agency's thinking about its standard solutions to a given problem.

We have learned from preparing a number of EIS's for EPA's sewage treatment projects that the planning constraints are seldom ideal and that the solutions one can choose are not perfect. These problems lead to a need for some creative thinking on how to mitigate the potential adverse impacts of a project, to provide greater benefits for clean water and the total environment. In addition to the creative thinking mandated by NEPA, the Construction Grants Program for wastewater treatment facilities encourages the use of new and advanced technology in projects which we fund (4).

We encountered a number of problems requiring creative solutions when preparing the EIS for new wastewater collection and treatment facilities at the Olentangy Environmental Control Center, in Delaware County, Ohio (5). The successful application of the EIS process is best illustrated in the series of decisions which lead to selecting innovative effluent disinfection.

Southern Delaware County is an attractive, semi-rural area located just north of Columbus, Ohio (see Figure 1). Our study area is divided by three major streams: the Scioto River, the Olentangy River, and

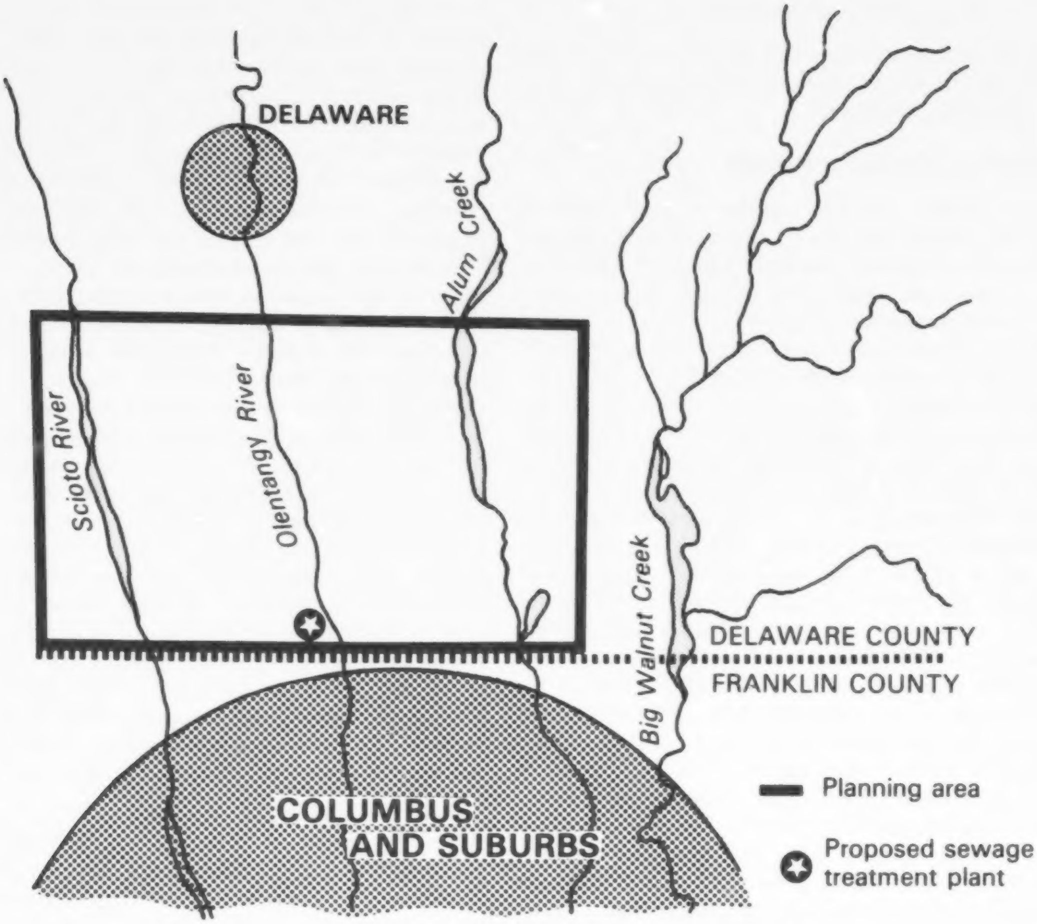

Figure 1. Regional Context for the Olentangy Environmental Control Center.

Alum Creek. These streams have been impounded at several points to provide reservoirs for water supply, flood control, and recreation. Local soil conditions severely limit the effective use of septic systems, while package treatment plants have been resisted by local authorities because of their poor maintenance history.

A proposal for Delaware County's central sewer service was presented to EPA in the 1974 Facilities Plan[1]. A 5,700 m^3/d (1.5 mgd) design size, two-stage activated sludge plant, with tertiary sand filters, phosphorus removal, and chlorination was to be built on the Olentangy River immediately above the county line. Known as the Olentangy Environmental Control Center, it was proposed to be expanded to 11,000 m^3/d (3 mgd) design size by the end of the 20-year planning period, to serve an expanded sewered area within south-central Delaware County.

EPA decided to prepare an EIS on this project in March, 1975. Key issues included the secondary impacts of the population growth to be induced by the plant, the facility's impacts to an adjacent regional park, its potential impacts to archaeological resources, its impacts to the Olentangy River, and public controversy[2].

The Olentangy Environmental Control Center presented a complex problem, for all of these reasons. In this paper, we will focus on its potential instream impacts and how we resolved to mitigate them.

[1] Burgess & Niple, Ltd. 1974. The sanitary sewerage facilities plan for south-central Delaware County, Ohio. Burgess & Niple, Ltd. Columbus, Ohio.

[2] Letter from Harlan D. Hirt. U.S. Environmental Protection Agency, Region V. March 28, 1975. Notice of intent to prepare an environmental impact statement, Delaware County, Ohio, and environmental impact appraisal.

MATERIALS AND METHODS

The Delaware County EIS was prepared by Region V of EPA, with the assistance of Enviro Control, Inc., of Rockville, Maryland[3]

RESULTS AND DISCUSSION

The scope of the EIS included an examination of existing stream quality and of the environmental impacts of centralized wastewater treatment. Additional information and public input about the impacts of the proposed wastewater treatment facilities arose during the EIS preparation period. The Olentangy River was designated a State Scenic River and provided both a valuable natural habitat and a good recreational resource. Although some impacts to it were noted from the existing upstream Delaware County wastewater treatment plant, (which is in the process of planning new improvements), the stream segment above the proposed effluent discharge location had recovered from the effects of the upstream effluent. Dissolved oxygen levels were good (5). Aquatic insects in this stretch of a basin-wide survey showed a high diversity, with about 70% classified as "pollution sensitive" (5). A great variety of mollusks have been found in the Olentangy River, including four state endangered species (5). A fishing census by the U.S. Fish and Wildlife Service demonstrates both the biological and recreational quality of the Olentangy as a warmwater fishery (5). The Olentangy River has regional significance because of its biological diversity, and because of the limited number of relatively natural stream segments remaining in central Ohio. Reservoirs, channelization, and pollution have gradually eliminated many high quality areas of running water.

Our first major decision in the EIS was the location of the treatment facilities (5). Were there any satisfactory alternatives to the one proposed in the Facilities Plan? Examination showed that the other two streams in the service area were not as suitable for effluent discharge. The Scioto River has downstream reservoirs for domestic water supply, while Alum Creek has an even lower average streamflow than the Olentangy. Using other sites along the Olentangy would have led to considerably higher construction and operation and maintenance costs. In addition, many of the Olentangy sites would use similar outfall locations. Moving the outfall downstream, below the end of the State Scenic River Segment, was proposed in the draft EIS (5). It was eliminated in the final EIS because of engineering complications, higher construction costs, and a substantial outcry from downstream residents. Regionalization of sewage facilities with Columbus was not possible because the long-term capacity of the existing interceptors was uncertain, and because of a variety of political and institutional problems. The original treatment plant site was chosen in the final EIS as the most feasible (5).

Within the constraints of using this site, we needed to decide how to provide wastewater treatment with a minimal amount of damage to the Olentangy River's ecosystem. We wanted a facility that would be cost-effective in the fullest sense. The effluent to be discharged from the plant, as originally proposed, would have had a chlorine concentration limit established by its discharge permit of 0.5 mg/l (5). At average streamflow, 12.5 m^3/s (441 cfs), this would dilute to an instream concentration of 0.003 mg/l for the 5,700 m^3/d facility. The extremely low flow condition, 0.13 m^3/s (4.73 cfs), based on the minimum release guaranteed from an upstream reservoir, would give an instream chlorine concentration of 0.17 mg/l for the 5,700 m^3/d (1.5 mgd) plant and 0.25 mg/l for the 11,000 m^3/d (3 mgd) facility (5). These concentrations assume no instream chlorine residual from the Delaware City effluent which is discharged upstream, so the actual instream values could be higher.

While we were preparing the EIS we consulted Dr. William Brungs of EPA's National Water Quality Laboratory at Duluth, Minnesota, about the effects of chlorine on freshwater life. He indicated that freshwater fish are extremely susceptible to chlorine concentrations during long term exposures. He recommended a total residual chlorine limit of 0.003 mg/l to protect aquatic life. Both coldwater and warmwater fish species were affected by concentrations above this limit.[4] The 0.003 mg/l maximum for receiving waters had also been recommended in the National Academy of Sciences' *Water Quality Criteria* publication in 1972 (6). EPA's 1976 Quality Criteria for Water Publication (EPA-440/9-76-023), which was published and distributed after production of the EIS, has since recommended a criterion of 0.002 mg/l for protection of Salmonid fish and 0.010 mg/l for other freshwater and marine organisms. At the average streamflow, at the 5,700 m^3/d (1.5 mgd) size, the Olentangy treatment facilities could discharge enough chlorine to reach this concentration within the com-

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[3] L. Peltier *et al.* 1975. Analytical studies for assessing the impact of sanitary sewage facilities of Delaware County, Ohio, Enviro Control, Inc., Rockville, Maryland.

[4] Memorandum from William A. Brungs, U.S. Environmental Protection Agency. February 19, 1976.

pliance limit of its National Pollution Discharge Elimination System (NPDES) permit. Because of this problem of toxicity to streamlife we decided that chlorinated effluent would be an inappropriate addition to this high quality segment of the Olentangy River.

At this point we obtained further technical assistance from the Wastewater Research Division of EPA's Municipal Environmental Research Laboratory in Cincinnati. We asked them to evaluate several disinfection alternatives, to decide if there was a better choice than chlorination. Their report confirmed the need to use a disinfection method that would produce less environmental damage than chlorination. Earlier demonstration studies performed by EPA researchers at Wyoming, Michigan, showed that effluent dechlorinated with sulfur dioxide and post aerated was nontoxic to freshwater biota. The same study concluded that ozonated effluent was also nontoxic. Both disinfectants were equally effective in treating tertiary effluent to meet the bacterial standard for disinfection. Other disinfectants, such as bromine chlorine and ultraviolet light, were considered too new and unproven, at that time, to be used for this application.[5]

Treatment costs were essentially similar for either dechlorination or ozonation, although ozonation was slightly more expensive. Dechlorination does not remove chlorinated organics, which have been of concern in drinking water supplies. Dechlorination also involves storing and handling chlorine and sulfur dioxide, which are both toxic chemicals. Ozonation offers several advantages to dechlorination. It has been used on drinking water supplies for a number of years with no observable toxicants. Ozone is produced as needed, and is not stored at the site.[6] Finally, ozonation does not increase the instream levels of dissolved solids (5).

CONCLUSIONS AND SUMMARY

We realized that ozone was a relatively new wastewater disinfection technology. At that time it was being used at two full scale treatment facilities, in Ohio and Colorado, while other systems were at the design or bid stage.[7] We wanted to design a wastewater treatment facility that would preserve and enhance the natural quality of this part of the Olentangy River. The mandate of both NEPA and the Construction Grants Program for innovative solutions to pollution problems led us to investigate alternate disinfection technology.

While ozonation costs were higher than those for chlorination, the definition of a "cost-effective" solution includes social and ecological factors, as well as monetary costs (4). Social and ecological considerations were significant in selecting ozonation as the cost-effective alternative. One of the major conclusions made by the Regional Adminstrator in the 1976 final EIS was to employ ozonation at the Olentangy Environmental Control Center (5).

Since then the facilities have been designed and the new wastewater system is presently under construction. Capital costs for ozonation in 1978 are about eight times higher than the 1975 estimated costs for chlorination.[8] However, this cost for ozonation is less than 1% of the total costs of constructing the first phase of the interceptors and treatment plant for south central Delaware County.

The EIS process was a successful tool for examining disinfection methods and choosing the one which best met local needs. In order to do this we had to take the time to investigate a variety of alternatives and to assess the environmental implications of our routine funding of chlorination for effluent disinfection.

The need for a thoughtful and creative approach to resolving water pollution problems applies to Facilities Planning as well as to Environmental Impact Statements. It is stimulating to attend a symposium, like this one, which presents the latest developments in a particular technical field. It is even more challenging to try to consider and appropriately use this new technology. I hope we can all work together to solve pollution problems and to reduce the environmental damage of human activities.

REFERENCES

1. National Environmental Policy Act. 42 U.S.C. ''4321 *et. seq.*

2. One of the most succinct sources for NEPA developement is the Annual Report series of the Council on Environmental quality; see Council on Environmental Quality. 1977. Environmental Quality-1977. U.S. Government Printing Office, Washington, D.C. pp. 116-129.

3. Council on Environmental Quality. 1978. National Environmental Policy Act. Proposed Regulations, 40 C.F.R. '' 1500-1508. Federal Register v. 43 no. 112. Friday, June 9, 1978. Part II. pp. 25230-25247.

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[5] Memorandum from E. F. Barth, U.S. Environmental Protection Agency. June 1, 1976.

[6] *Ibid.*

[7] 40 C.F.R. '35, appendix A.4.f.3.

[8] Personal communication with Gary Gilbert, Delaware County Sanitary Engineer. July, 1978.

4. Environmental Protection Agency. 1974. Final Construction Grants Regulations. 40 C.F.R. • 35.908. Federal Register v. 39 no. 29. Monday, February 11, 1974. Part III. p. 5256.

5. U.S. Environmental Protection Agency, Region V. 1976-b. Final environmental impact statement for the Delaware County, Ohio Board of Commissioners Olentangy Environmental Control Center and interceptor system. U.S. Government Printing Office, Chicago, Illinois.

6. National Academy of Sciences - National Academy of Engineering. 1972. *Water Quality Criteria 1972*. USEPA, Washington, D.C. 594 pp.

DISCUSSION

QUESTION: Did I understand you correctly to say that the average flow of the river is normally used in Ohio to calculate the dilution factor for a disinfectant?

MR. MACMULLEN: No, you did not. I said that the average flow was documented at 441 cubic feet per second. I gave what the chlorine residual would be from a 1.5 and a 3.0 mgd facility at that average flow. But then I said, "However, this is not a free flowing stream." Over and above the average and low flow, you have the problem that this is a regulated stream and in fact, the critical low flow that we use for planning purposes is 4.73 cfs.

QUESTION: I realize that in this case. But in other cases where you do not have a regulated stream, would you use the average flow?

MR. MACMULLEN: No, sir. We use the seven day, once in ten year low flow.

DR. RICE: I am fascinated by your thought processes. I think this is the first ozonation sewage treatment plant that I have heard of in which ozone was chosen without apparently doing any piloting work. Is that really the case?

MR. MACMULLEN: We did not do any piloting work. That is true.

DR. RICE: There was not a collection system?

MR. MACMULLEN: That is a good point. There was no collection system there to begin with, so it would have been somewhat difficult to do pilot work in southern Delaware County. I presume that we could have gone to the city of Delaware and established a small pilot facility there. The applicability of results from the city of Delaware to the Delaware County sewage treatment plant would not be 100 percent necessarily, because there are industrial wastes in the city of Delaware which we did not anticipate in southern Delaware County. But in answer to your question, we did not do any piloting work.

DR. RICE: I was not being critical at all. It just seems to me it is a famous first for ozonation. That is all.

MR. MONTAGUE, *EPA* (*Philadelphia, Region 3:*) I just have a comment. I think the meeting has been very enlightening in terms of what information is now available, and I have learned a great deal from the many speakers and information that has been provided. But frankly, I am also left in a quandary, and that is to say, which system is most applicable to a given situation. Since the matrix is so complex, we find that we really have to look at it very, very uniquely for each and every case.

But I think what is really needed for the engineer, and particularly for the state and local entity that is going to have to sign off on a process, is to have some basis of comparison initially to make some judgments as to which direction they would really like to go. I think that the system right now as it stands leaves us somewhat puzzled as to which one is really cost effective based on what standard.

The comment that was made several times during the last few days about "what is disinfection" I think really comes to roost here. I would like to suggest to Al, I think the region would support him in this regard, is that we go forward with a joint effort on the part of not just one company but on the basis of the various companies that are here represented and those that are not here in the ozone, UV, or chlorine entities and to present, if you will, a project that would at least on some equal level make a determination of what is the cost effectiveness in a given situation. I realize there will be departures from that. But at least it would give the engineers and those who are signing off on a contract some insight as to what might be the situation.

I know many may not agree with it. It is one man's opinion. Thank you.

MR. WHITE: What is the disinfection requirement and who set it?

MR. MACMULLEN: The disinfection requirement was set in the Ohio Water Quality Standards that we had to deal with. And that was 200/400 coliform bacteria.

28.

THE CONSULTANT'S VIEWPOINT ON ALTERNATE DISINFECTANTS

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INTRODUCTION

Disinfection of wastewaters for the control of enteric and disease-producing organisms has been common throughout the United States for a number of years. In almost all instances, the mechanism for disinfection has been the application of chlorine to treated effluents. Until recently, little consideration was given to the possible hazardous effects of effluent disinfection on the environment. Concern about these effects has been expressed in recent years, however, and the use and availability of other disinfectant processes is now being more thoroughly investigated. In light of these developments, it has become increasingly important for the engineering consultant to formulate an evaluative procedure for the assessment of disinfectant use in any given situation. This procedure must take into account not only the relative merits and disadvantages of chlorine use, but also those associated with use of other disinfectants. This paper presents such a decision making model along with an actual case study of its use.

AVAILABLE ALTERNATIVE DISINFECTION TECHNIQUES

Wastewater disinfection processes, as they are utilized today, involve specialized treatment for the destruction of harmful and otherwise objectionable organisms. Classically, disinfection processes have been employed for the purpose of destroying or inactivating disease causing microorganisms, primarily bacteria of intestinal origin, contained in domestic wastewater. Other organisms which are currently the target of disinfection processes include viruses, intestinal protozoa, and various microorganisms.

The advantages and disadvantages related to the use of alternative disinfectants or disinfection techniques should be considered according to the following factors:

- Technological state-of-the-art.
- Effectiveness in destroying particular types and numbers of organisms under certain environmental conditions (pH, temperature, etc.).
- Safety precautions necessary for transport, storage and application.
- Costs associated with physical facilities and materials.
- Effects of reaction by-products.
- Availability of practical, accurate assay techniques for determining disinfectant concentrations.
- Tendency of the disinfectant to persist in residual concentrations.
- Energy requirements.

Most of the disinfectants currently being considered for domestic wastewater treatment have one or more serious limitations that preclude their general acceptance and adaptability. The next section of the paper deals solely with the disadvantages of chlorine for disinfectant use.

THE NEED FOR ALTERNATIVES TO CHLORINATION

The use of chlorine to disinfect wastewaters has undeniably produced a public benefit of great magnitude. Wastewaters contain a number of infectious microorganisms, including *Shigellae, Salmonellae,* and enteric viruses, that are removed by disinfection. In the majority of cases, this disinfection has been accomplished through the use of chlorine. Reductions in the outbreaks of waterborne diseases have been apparent since its widespread use. It is obvious that chlorine disinfection has served to protect the public health in the past; however, it may have also contributed to potential environmental damage.

In recent years much attention has been focused on the occurrence of certain halogenated organics in drinking water supplies, both in this country and abroad.

Technological advances have produced instrumentation sophisticated enough to measure part per billion quantities of chemicals formerly unknown in domestic water supplies. Public attention was brought to this fact when, in November of 1974, the United States Environmental Protection Agency announced the presence of small quantities of 66 organic chemicals, some of which were suspected carcinogens, in the New Orleans water supply. Since then, similar discoveries have been made in all parts of the country (4).

Several of the compounds that are suspected of being carcinogenic or toxic have been shown by numerous researchers to result from disinfection with chlorine (1, 3, 6). Of these, the following have been found in chlorinated wastewaters: methylene chloride, chloroform, trichloroethane, trichloroethylene, dichlorobenzenes, trichlorobenzenes, and tetrachloroethylene. Because of the health hazard associated with the ingestion of these and other halogenated organics, chlorine can no longer be considered the optimum disinfectant for all uses.

If chlorine is no longer regarded as the best possible disinfectant, other alternative disinfectants must be considered in the planning of wastewater treatment systems. These considerations should be based on factors such as the potential uses of the water and applicable state stream standards. Several questions that may be asked in determining the best disinfectant for a particular installation include the following:

- Is the stream inhabited by sensitive aquatic life? (Chlorine residuals may prove toxic to many aquatic species.)
- Will the water be used for human consumption? (The presence of the halogenated organic compounds may be possible.)
- Will body contact recreation take place within the water? (Disinfection of effluents is paramount if the answer is yes.)
- Are there in-stream standards regarding chlorine residual? (If so, dechlorination of effluents may be required where sensitive aquatic life exists.)

The appropriate place to ask these questions is in the preliminary planning stages of treatment plant design. This is best accomplished as part of the 201 Facilities Planning Process outlined in the next section.

CONSULTANT'S PLANNING APPROACH TO EVALUATING ALTERNATIVE DISINFECTION TECHNIQUES

From a consultant's viewpoint, the selection of appropriate disinfection techniques for municipal wastewater treatment facilities must be accomplished through the 201 Facilities Planning Process. The overall goal of this planning process is to develop a strategy based upon the most cost-effective solution which will meet established effluent and water quality goals within a particular planning area while accommodating recognized environmental and social contraints.

With respect to wastewater disinfection, feasible alternatives are identified on the basis of effluent limitations and water quality standards which are usually based on designated water usage and differ from state to state.

The feasible disinfection alternatives are then technically evaluated by the consultant in terms of the following criteria:

- Water quality impact
- Monetary cost
- Implementation feasibility
- Environmental effects
- Resource and energy usage

It should be pointed out that the resource and energy usage requirements are evaluated primarily in terms of cost. The preferred disinfection alternative should then be determined by the consultant with input from a Facilities Planning Advisory Committee (composed of local officials, citizens, regulatory agency representatives, and environmental groups) which assigns an overall priority to each disinfection alternative based upon a criteria rating system established or approved by the committee. This committee process insures public participation in the development of an over all facilities plan.

In the past, the 201 Planning Process has usually led to the idenfication of liquefied chlorine gas or sodium hypochlorite as being the most suitable disinfectant according to the established criteria and priority ranking system. In light of recent findings concerning the environmental effects of these disinfectants, a reevaluation of priorities appears to be warranted. A suggested procedure for evaluating alternative disinfection processes during the planning process is presented in the form of a decision tree shown in Figure 1. As indicated, this decision process is based

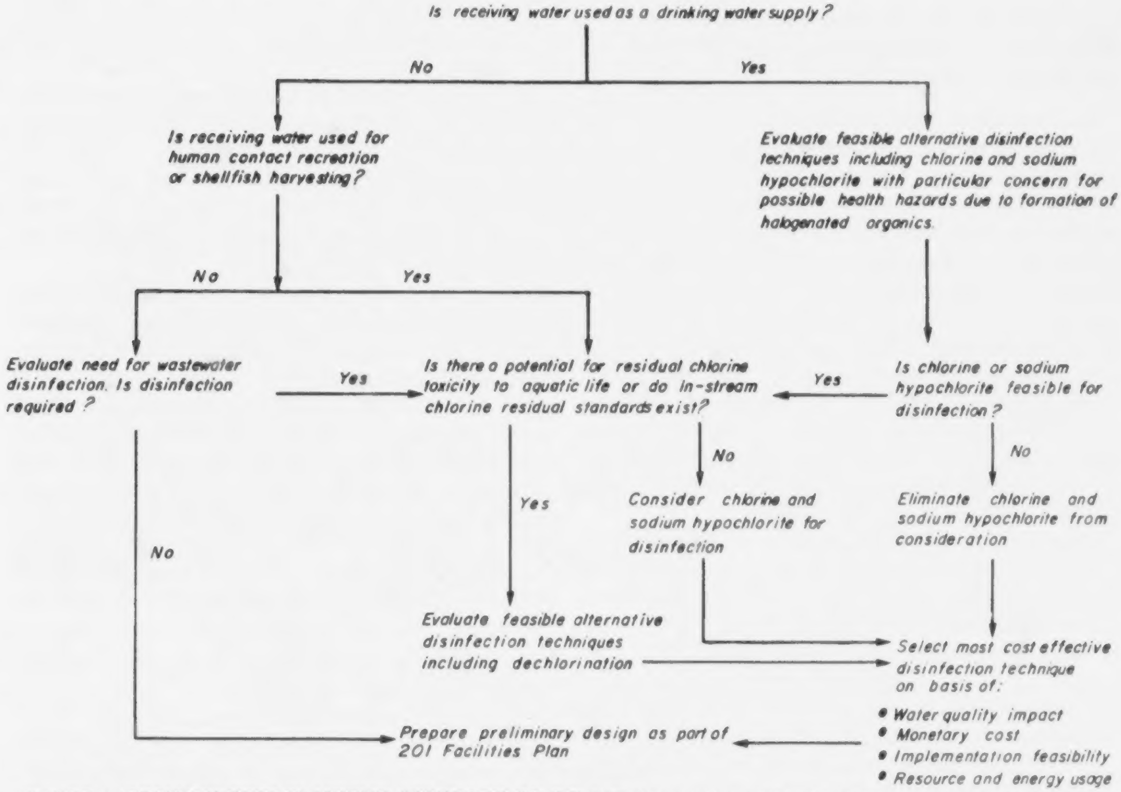
Is receiving water used as a drinking water supply?
No
Yes
Is receiving water used for human contact recreation or shellfish harvesting?
Evaluate feasible alternative disinfection techniques including chlorine and sodium hypochlorite with particular concern for possible health hazards due to formation of halogenated organics.
No
Yes
Evaluate need for wastewater disinfection. Is disinfection required?
Yes
Is there a potential for residual chlorine toxicity to aquatic life or do in-stream chlorine residual standards exist?
Yes
Is chlorine or sodium hypochlorite feasible for disinfection?
No
No
Yes
Consider chlorine and sodium hypochlorite for disinfection
Eliminate chlorine and sodium hypochlorite from consideration
No
Evaluate feasible alternative disinfection techniques including dechlorination
Select most cost effective disinfection technique on basis of:
• Water quality impact
• Monetary cost
• Implementation feasibility
• Resource and energy usage
Prepare preliminary design as part of 201 Facilities Plan

FIGURE 1. DECISION TREE PLANNING APPROACH TO SELECTING A WASTEWATER DISINFECTION TECHNIQUE

upon anticipated water usage and the application of in-stream chlorine residual standards. Therefore, highest priority is given to the water quality and environmental effects with particular emphasis on eliminating the following:

- Health hazards caused by disease producing organisms.
- Health hazards caused by halogenated organic materials.
- Potential for residual chlorine toxicity to aquatic life.

The suggested decision process is based on the use of chlorine and sodium hypochlorite since these are the most widely used disinfectants and they are currently in the midst of considerable controversy. It is significant to point out that these disinfectants are not eliminated from consideration in the decision making process unless they are shown to pose a health threat to man, or if alternative techniques are shown to be more cost-effective. Considering current technology, the decision process will usually determine that wastewater discharges to large rivers, which provide sufficient dilutional capacity to minimize health hazards due to halogenated organics, can be most cost-effectively disinfected with chlorine.

An additional feature of the proposed decision tree is that it permits an evaluation of the need for wastewater disinfection in cases where the receiving water will not be used for drinking water, human contact recreation of shellfish harvesting. This feature is consistent with the conclusions of the U.S. Environmental Protection Agency Task Force (2) finding that in some circumstances, "such as high dilution and die-off, seasonal recreation, and no downstream reuse potential, the benefits of disinfection for protection of public health are minimized and may not be needed."

A major advantage to the suggested decision process is that it provides the consultant with a well defined approach for evaluation of alternative disinfection processes. It is anticipated that the utility of such an approach will increase as the state-of-the-art of alternative disinfection techniques becomes more established. A case study involving the use of a similar decision tree process is summarized in the next section.

CASE STUDY OF DECISION TREE PLANNING FOR ALTERNATIVE DISINFECTION TECHNIQUE SELECTION

A case study of the decision tree planning approach to the selection of the optimum disinfectant was conducted in Louisville, Kentucky, for the Louisville and Jefferson County Metropolitan Sewer District (5). In this study, a Welsbach ozone pilot plant was put into operation at the Fort Southworth Sewage Treatment Plant to evaluate the possibility of ozone use in disinfection. At the time the study was conducted, the Fort Southworth plant was a primary sewage treatment facility discharging to the Ohio River downstream of Louisville, Kentucky. Renovations were being planned to upgrade the plant to provide secondary treatment including disinfection, so the decision tree approach was utilized to aid in the selection of the disinfectant.

The Ohio River is used as a drinking water source; therefore, alternate disinfection methods were considered along with chlorine use. The dilution effect of the river made the use of chlorine a viable alternative for consideration. Of the available alternate disinfectants, ozone was chosen for further study since this process phased in well with the proposed high purity oxygen activated sludge treatment scheme, it had been used in Europe for disinfection, and the U.S. Environmental Protection Agency was interested in a pilot study involving ozone. The experimental approach was taken and results of the study are outlined below.

In preparation for the study, a 150 m^3/d (40,000 gpd) package plant was added to the $4 \times 10^5 m^3/d$ (105 mgd) Fort Southworth facility to provide secondary treatment prior to ozonation. This step was necessary to evaluate the use of ozone in the treatment scheme proposed for the plant after upcoming renovations. The ozone contact reactor was connected to the effluent weir trough of the package plant. The reactor consisted of nine rectangular chambers through which ozone was dosed at approximately 15 mg/l. Samples 5 to 10 ml in size were taken at influent and effluent points and in each chamber for viral and bacterial analyses. The results of the study were as follows:

- a 15 mg/l ozone dose provided 99+% removal total coliform, fecal coliform, and fecal streptococci bacteria. Average effluent concentrations after ozone treatment were:
    - total coliforms — 500/100 ml
    - fecal coliforms — 103/100 ml
    - fecal streptococci — 9/100ml
- Ozone provided nearly 100% inactivation of a test virus within a 5 minute contact time at a total ozone dosage of 15 mg/l and a residual of 0.05 mg/l.
- Oxone provided an average turbidity reduction of 70%, a COD reduction of 29%, and color removal.
- Ozonated effluent produced no noticeable harmful effects on native fish populations while non-ozonated secondary effluent proved toxic in a test study.

Obviously, ozone was proven to be an effective disinfectant for use in the Fort Southworth treatment scheme. The decision tree approach was utilized in the final disinfectant selection as outlined below.

As a result of the ozone pilot study, both ozone and chlorine were being considered for use as a disinfectant following the high purity oxygen secondary treatment process. In the case of chlorine, there were no water quality standards limiting in-stream levels and adverse effects on aquatic live were not expected. The costs for the ozone system were estimated to be slightly higher than for chlorine disinfection. However, it was also realized that direct comparison of process costs was not realistic since ozonation achieved effluent polishing in addition to disinfection. Nonetheless, at that point in time chlorine technology was well known while ozone use was rare in this country and a conservative approach was preferred. For these reasons, chlorine was chosen as the optimum disinfectant for use at the Fort Southworth Treatment Plant.

SUMMARY

The use of chlorine as a disinfectant in wastewater treatment systems may not always provide the maximum benefits to be obtained by bacterial reduction. Chlorine residuals have been noted to produce adverse effects upon aquatic life and to form several carcinogenic or possibly toxic compounds when applied to wastewaters. For these reasons the use of a number of alternate disinfectants must be considered in wastewater treatment systems. These include ozone, ultraviolet radiation, bromine chloride, chlorine dioxide, and dechlorination methods. The availability of various disinfection alternatives necessitates the use of an appropriate decision making process to provide a mechanism for selection of the optimum disinfectant. The decision tree approach, presented herein, considers both the environmental and health effects of disinfectant use for the evaluation of various disinfection alternatives. Use of such a technique should avoid decisional paralysis and incorporate disinfectant selection as a logical part of the facilities planning process.

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DISCUSSION

MR. WHITE: Was that ever characterized as a well oxidized secondary effluent?

DR. PAVONI: It was a fairly well oxidized secondary, but it was not filtered which obviously it now would be.

DR. TITTLEBAUM: There was a nine hour detention time in the secondary followed by two clarifiers in series.

MR. WHITE: Was it nitrified?

DR. PAVONI: No. I think that a point to make here if I remember right, is the estimate of overall treatment costs at that time at that facility was 20 to 25 cents per thousand gallons. The cost of chlorine was estimated at somewhere around 1.0 to 1.5 cents per thousand. The cost of the ozone was estimated somewhere around two to three times that. So it was two or three times the disinfection costs, but viewed in terms of the overall treatment costs, it was not a significant percentage change. I think that if that same pilot study conducted in 1972 was conducted today with the concerns about the organics in the Ohio River, probably ozone would be the disinfection process at that plant instead of chlorine.

MR. WHITE: I just want to make a comment that I mentioned to Joe in private. We have the decisions in California already made for us. But this is very interesting because it took California about 30 years to finally wind up with this type of process. So every discharger in the state is faced with a given set of conditions that is already spelled out for them in this selection of what the requirements will be.

MR. FLEISCHER: I just have a quick comment in that I am somewhat familiar with this through the person who ran the pilot plant, Carl Nebel. You did not really go over anything but disinfection, and I realize this is a disinfection seminar. But if anybody did see that froth forming, and I heard some comments, that was what I think you refer to as the polishing, which is suspended solids removal. This is another aspect of ozone we cannot really discuss here in disinfection. But in some cases when you have a low amount of suspended solids, ozone will remove it perhaps better than a filter will. That is the froth you saw at the top of the contact chamber.

DR. PAVONI: Right, that is very true. We did not get into the other aspects besides disinfection.

DR. RICE: I have a quick observation on this. The contacting system that was used in Louisville was a nine chamber bubbler, which effectively showed that after the third chamber, you really did not need the other six. This was actually confirmation of the earlier Blue Plains study in D.C. which also used a nine-chamber contactor and also showed that after the third one, you do not really need the other six. Now five years later, Estes Park puts in a nine chamber bubbler contactor. I am not really criticizing other than to say that someday I think we ought to learn from what we have done in the past. I think that was an expense that did not have to be.

DR. PAVONI: Since there are no other questions, one final comment is that I realize that California in most cases is ahead of the rest of the country as far as implementing environmental controls and procedures. All I think we are getting to here in our paper is the fact that we work primarily in the midwest, having offices in both Louisville and South Bend. We really have not seen this type of approach reviewed by either the state agencies or by Regions 4, 5 or 7, in general. We just think that it is something that should be emphasized a little more strongly, especially now with the increased emphasis that is being placed on disinfection.

MR. BARTH: Thank you, gentlemen. We have an unscheduled speaker, Jack Mills from Dow Chemical. Jack, we are going to let you have five minutes.

DR. MILLS: There have been a number of people who have asked about bromine chloride. Since we were not on the program, I thought I might spend a few minutes just discussing what has happened since the Wyoming project. Some of the things I am going to say quickly here probably should have been said before lunch. It might have fit in with some of those other subjects. One of the reasons is that bromine chloride is a very effective virucide and it has been shown to be better than chlorine. This is one of the advantages that it has had.

In the Wyoming project, as some of you know, they looked at alternatives to chlorine; ozone, bromine chloride, and dechlorination were the three alternatives. The conclusion from that trial indicated that chlorine indeed was a problem, both chronic and acute toxicity-wise, whereas the alternatives (ozone, bromine chloride, and dechlorination) were not, as far as toxicity.

As far as antimicrobial activity or control of microorganisms, both dechlorination and bromine chloride were satisfactory on secondary effluent and ozone on tertiary effluent. Since then we wanted to conduct some full-scale plant studies. Some of you know we had some problems with the feeding equipment at the Wyoming project and we had the same problem in our first trial. But I am confident we ironed that problem out. Bromine chloride is a liquid. It has to be vaporized, which poses a unique problem. It does feed very similarly to chlorine and you can use similar equipment, but you do have to use a vaporizer and that was our main problem.

In our trials we picked both secondary and tertiary treatment plants. I think mainly secondary is important because I would guess most plants do not produce filtered nitrified effluent in this country nor even primary as mentioned earlier. But to evaluate the suspended solids problem and this sort of thing, you really have to look at a typical secondary plant.

Anyway, we have two plant trials going right now — one in Hatfield Township, Pennsylvania, and this is a 3.6 mgd tertiary treatment plant; and then we are also involved with the state of Maryland on a 3 mgd plant at Sykesville, Maryland. This is a secondary treatment plant. Both of them are activated sludge.

These trials have been running several months. The data so far indicate that we do control the fecal coliform levels between zero and 20 per 100 ml as well or better than chlorine was at lower dosages. In both cases, I think the dosage was half the chlorine dosage.

The residuals are running in the neighborhood of 0.1 mg/l at the outfall, but shortly after downstream from the outfall, they disappear completely because bromine chloride residuals continue to drop off. This is the one significant difference between chlorine and bromine chloride, that is, that you form an unstable residual. The bromamines decay much faster than chloramines and disappear quite rapidly. So we are meeting those requirements.

The third thing I want to mention is that we are looking at organics. Now one of the things that I think is very important here is that haloforms do not seem to be formed as much in sewage disinfection as they are in potable water, and there are reasons for this. Now I will not get into this, but we are looking at haloforms in the outfalls of these plants and we are finding less than 20 parts per billion. Shortly downstream there are none observed. The reason for this from our laboratory data is that the haloforms in wastewater effluent vaporize or are volatile and disappear. That is not the case in potable water. In potable water you have a pressurized system and the chloroform continues to build up in the distribution system. When you drink a glass of water, you are going to get so many parts per billion of chloroform. If you drank out of a stream down from a sewage plant, I am confident that you will not see any chloroform because the half-life in water is less than six hours.

We are also looking at nonvolatile organics. Bob Jolley did not mention this, but Ethyl Corporation ran a pilot study down at Hampton Roads, Virginia. They evaluated some of their effluent for brominated organics. But it does indicate that the brominated compounds are much less active or much less stable, and that is one of the key differences in the halogen chemistry between chlorine and bromine.

First of all, oxidation is the predominant reaction. And second, the types of compounds that are formed, are very low in concentration and are much less stable.

MR. WHITE: Jack, when am I ever going to sell you on the idea of taking existing chlorinator installations, use the primary chlorinator to satisfy the initial demand, and then generate bromine on site with the stand-by chlorinator and bromide salt. You would have the whole thing made, instead of going through all this rigamarole?

DR. MILLS: George, we have considered that and there are a couple problems. We have tried it. One problem is that it is more expensive. It is not as simple as you say.

MR. WHITE: It is simple in France.

DR. MILLS: Well, ammonia interferes with that reaction between bromide and chlorine. And you do have a problem of mixing and getting a complete reaction. I am not saying it is not possible. I know the French have done it.

APPENDIX A.

BANQUET PRESENTATION:

"THE LAW'S RESPONSE TO PUBLIC HEALTH HAZARDS"

J. V. Karaganis

Karaganis and Gail, Ltd.
Chicago, Illinois

MR. VENOSA: Mr. Francis T. Mayo, who is the Director of the Municipal Environmental Research Laboratory here in Cincinnati, will be introducing our guest speaker, Mr. Joseph V. Karaganis, attorney from the state of Illinois.

Mr. Mayo has been our Laboratory Director for approximately two and a half years. Before he was appointed director of MERL, he was Regional Administrator for EPA Region 5 in Chicago for six years. During that time, he became very well acquainted with Joe Karaganis. So I think it would be fitting for Mr. Mayo to introduce Mr. Karaganis this evening.

MR. FRANCIS T. MAYO: This is indeed a privilege. I am going to call him Joe, not Mr. Karaganis, because I have known him as Joe for eight years now.

We were reflecting on the occasion when Joe and I first met. It was Earth Day, Chicago, 1970, at a time when Joe Karaganis was kind of downy-cheeked and I was a very virginal Regional Administrator for EPA.

Many things have happened in the ensuing years. Joe enjoyed a very enviable position for an attorney, I thought, during those years because of a special assistant relationship to the attorney general in Illinois. The attorney general's office there enjoys some environmental latitudes that most attorney generals do not. The office has the opportunity to initiate actions of its own whereas in most states, they come to the attorney general's office through an environmental agency.

Bill Scott in Illinois has had a deep personal concern for environmental issues and a pretty astute political perspective. With Bill Scott's initiative and Joe Karaganis' talents, they have built a very, very enviable reputation for dealing with the giants in industry and taking on the big ones in some very, very difficult and very challenging environmental issues. Through the years between them, they have enjoyed an outstanding reputation for successes, one of the most recent ones being the increasingly famous City of Milwaukee litigation in which the state of Illinois initiated suit in the federal district court to bring about some abatements and improvements in the management and operation in the overall facilities for the Milwaukee wastewater treatment system as well as those for some of the intervening municipalities between Milwaukee and Chicago.

That decision has created great anxiety among many of the major metropolitan sanitary districts in the country. It has added to the commentary on the role of the courts in resolving difficult environmental issues. It has thrown a scare into many, many organizations faced with somewhat comparable circumstances.

In addition to being a special assistant to the attorney general in Illinois, Joe has a private practice — the name is Karaganis and Gail. As a firm, they are achieving an outstanding reputation nationally for taking on and prevailing in a number of very, very difficult environmental litigations. I am confident that you will find Joe's commentary on the Milwaukee case most interesting, and I think you will find him a very delightful speaker.

From an attorney's standpoint, Joe has a very, very great capability to be both a quick learner and to be able to translate basic principles of science and engineering experiences to the courtroom scene and to use the combination of those factors and the law very, very successfully.

It is my pleasure to offer to you as your speaker this evening Mr. Joe Karaganis.

MR. JOSEPH V. KARAGANIS: Thank you. Frank, ladies and gentlemen at the main dinner table and ladies and gentlemen in the audience.

As Frank suggested, at the time when he first met me back in 1970, I had been working in the field of pollution control a relatively short time. I think we have both received a few bruises and had a few experiences in that interval of time since then.

To show you how one accidentally, and literally accidentally, gets into a field like this, my first experience with pollution problems came from Congressman Abner Mikva from Illinois—some of you may know of him. He was at that time a congressman from the south side of the city of Chicago and asked me to go out and help some of his constituents who were having a fight with an asphalt batching plant. It was not a water pollution problem; this was an air pollution problem.

I went out there, and it turned out that the asphalt batching plant was not only using a sour crude or a sour petroleum derivative highly loaded with sulphur for its petroleum base for the asphalt, but they were also using slag from the Wisconsin Steel Company, which had again a great deal of sulphur in it, for the basis of the aggregate (the hard material that made up the asphalt). Presumably, this was all again theory. The heating up of this process, be it the sulphur and the oil or the sulphur and the slag, resulted in an intensely acrid smell throughout the residential neighborhood which was surrounding this plant. If you have ever stricken a match close to your nose and caught that stench, you know what I mean; only this was on an eight to ten to twelve hour a day basis.

This phenomenon had occurred in the way that Chicago politics traditionally worked these things. What had happened was that environmental control over the years had been one of zoning—to keep the offensive odors away from the residences. We do not work things that way in Chicago. As long as the industrial land next to this plant was vacant, some sharp real estate developer got ahold of his local alderman, the right amount of consideration passed between them, and the next thing you know we had zoning for residences right next to this asphalt batching plant.

The poor yokels who came out to buy their homes—and home ownership is a very serious and intense thing among many people in Chicago—came out typically on a weekend (on a Saturday or a Sunday) and were told by the honest and forthright real estate developer that the asphalt batching plant was closed and would never operate again. Lo and behold, upon coming into the neighborhood, they found somewhat to their chagrin that that was not the case.

Well, when I got there, I smelled this odor like they did. I reported it to the city officials, thinking, of course, we would get immediate and aggressive enforcement. Lo and behold, I still had a few things to learn about Chicago politics which, for all the jokes, we are not that different from the nation as a whole. I found that I did indeed get an inspector. An inspector from the local Air Pollution Control Department came rushing out in his car with his flashing Mars lights and it looked very official and very impressive. He sniffed the air and said, "I don't smell anything. If anything, it smells to me like bacon and eggs. It smells pretty good to me."

Well, I learned that this inspector subsequently had the name Gilfoil which was a very helpful name to have in Chicago at that time because Gilfoil was the maiden name of a woman by the name of Eleanor Daley, who was married to the then-mayor of the city of Chicago.

But as it happened, Inspector Gilfoil never did find a problem. Over a four year period, we had to take this asphalt batching company to the circuit court of Cook County and fight it through as both a citizen's suit and subsequently a state of Illinois suit, until we got relief. The asphalt plant really gave an example of how the law relates to environmental hazards. I could not sit back and tell you what public health hazard that asphalt company presented. I know from the standpoint of just a daily living aesthetic nuisance, it made it impossible to live in the area and enjoy your home at all. People would try and get out of the area as soon as they could. But it gave a good example as to the step-by-step process that one faces when trying to present this problem to either public agencies for administrative correction or to the courts for judicial abatement.

One of the things I remember is that the first step is always: "Your Honor, we're not causing any problem. These people are crazy." Finally the court would be convinced after witness after witness would come on the stand that yes indeed, there was a problem.

The next thing was to look for the quick and dirty cure. And the quick and dirty cure—and you can pick your field, be it water pollution or air pollution—is always something that is very low cost and the perpetrator of the act will say "we will immediately and permanently cure the problem."

It was kind of funny and sad in a way. This fellow who ran the asphalt plant, which interestingly enough we subsequently found had a lot of contracts with the city of Chicago, which may have had something to do with why we were getting less than aggressive enforcement, decided that he was going to solve the problem by putting perfume into the smokestack.

You can imagine the feelings in the neighborhood when in addition to the acrid, bitter, biting stench of the sulphur that this was overlaid with an incredibly heavy scent of what smelled like soapy perfume throughout the neighborhood. That was not either the immediate or the ultimate solution to the problem.

But my remarks here tonight are not to deal with aesthetic nuisance, although that can be a significant one. I would like to deal with the question of (and it was a topic that I suggested) the law's response to public health hazards, and in particular the law's response, be it judicial or administrative, to the question of the public health threat presented by pathogenic organisms in human sewage.

One of the things that I asked when I came here this evening was to what extent your seminar and conference had discussed this issue. One of the things that I find incredibly frustrating in the field today is almost a schizophrenia within the Environmental Protection Agency as to what should be the control, be it intermittent or permanent or nonexistent, at the discharge point of sewage treatment plants; hopefully it gets to the sewage treatment plants. In most cases it does not. I will have a little bit to say about that—what should be the level of control for pathogenic discharges?

I happen to be a public health advocate. I have yet to find the expert today anywhere who can sit back and say to me that I can safely rely upon the forces of nature to correct the problem, or to rely upon the municipal officials to take the water in and to treat it so it will not cause any hazards, or who can give the kind of guarantees that I feel I am entitled to as an average citizen in consuming public waters as a resident of the Chicago metropolitan area or any other area. And I am an advocate for maximal controls, maximum elimination of pathogenic organisms at their source. Containment is a philosophy that I have followed as a lawyer not simply by one of legal or political philosophy but one of having cross-examined probably three to four hundred experts in a whole variety of fields and finding and being amazed by and being impressed by not how much we know but how little we know with respect to the receiving environment's capability of assimilating various pollutant or contaminant discharges. You would be amazed when you start going down the track of variables that exist in nature as to how little the most prominent experts in the field can tell you about the variability of the concentration and strength of the discharge, about the efficacy of the treatment methods that currently exist, about the transport and diffusion and reconcentration of pollutants in the ambient or receiving environment.

So what I am suggesting to you is I want to get it clear from the start that I have a bias in favor of containment. But I find frustrating the agency to which we look for guidance, the big eagle I call it. . . .

Now I will tell you, it is a great game that lawyers play, a great game. How many of you have ever seen a report that is not worth the paper it is written on but it has a big eagle on the front? "United States Environmental Protection Agency." Lawyers love to wave those things. They walk in, "Judge, the United States Environmental Protection Agency says. . . ." What is a problem is when both lawyers are waving documents saying, "United States Environmental Protection Agency. . . ." and they say diametrically opposed things. And this is one of the problems we are having in the field of pathogenic control today.

I cannot identify the source of this dilemma. I do not know in the field with the public health professionals that I talked to, they are almost universally in favor of strong controls. In those of whom have said, "Don't worry. There isn't a threat," I have found they have not done the level of research in many instances which allows them to establish definitively that there is not a threat. I would accept immediately anybody's statement that: "Yes, there are a lot of unknowns." But for people to say, "Don't worry about it," is to me totally unfounded from a standpoint of any empirical evidence that I have been able to see.

And I think this dichotomy between "We don't know; therefore, it won't hurt us," and on the other side, "We don't know all the answers; therefore, contain"; is one that has been reflected in the development of the law's response to public health hazards.

Many of you have heard about nuisance law as being relatively antiquated and not able to respond to current needs. Let me suggest to you that much of the development currently within the field of environmental law is based upon the identical analysis of public health risk and public health risk containment that is being developed in the most sophisticated of recent cases.

When you go back to (and we recently did this in the Milwaukee case) the federal Appeals Court in Chicago, which is now hearing the Milwaukee appeal, they said to us and to Milwaukee, "Really tell us what common law nuisance is all about." Everybody thinks they know what nuisance law is all about. Most laymen do; most lawyers do. We take in the course called "Torts" and we read a hornbook statement of what a nuisance is, but we never really have analyzed what its origins were and why it developed. I would like to if I can because this is very, very important.

How many of you in dealing with lawyers in any of these fields have heard the statement by the lawyer, "Well, we cannot stop this problem unless we have clear and convincing evidence that it will certainly cause harm." This is the standard defense line in any legal attack, be it quasilegislative through a regulatory process, be it through a permit process, or be it through a judicial attack. You must show clear and convincing evidence that there will be harm. This so-called clear and convincing burden turns out to have been developed through an anomaly, a quirk, in the jurisdictional development of the law between what are known to lawyers as the fields of law and equity. I would like to tell you about this in a little while because I think it will convince you that the law offers remedies that most of you may have felt in the past have not been available.

We went back as a result of the mandate from the 7th Circuit Court of Appeals and we went back to such publications. I will tell you that I had the fortunate or unfortunate experience of attending a Jesuit secondary school and I could not remember one ounce of my Latin, but what I could remember I believe were Cicero and Caesar's commentaries. But what I could not remember and could not decipher at all were the medieval bastardization of Latin that came across in such publications which we had to go to. We had to go to the Latin translation of such things as "Lord Glanville—A Treatise on the Law and Customs of the King of England" which was written in the year 1189 A.D.

Now in Glanville and books like "Coolie's Treatise on Blackstone's Commentaries," and "Hawkins' Pleas to the Crown," and Stephen's "A Disgest of the Criminal Law," we were able to gather the roots, the origins of the field of common law nuisance, particularly with respect to the public health hazards and the English response to those public health hazards. Why are we so concerned about the English response?

Well, we lawyers love to cloak ourselves in these broad, amorphous things. I am always critical of scientists about using terms like "judgment" rather than being precise and finding out what is behind the judgment. Well, we do the same thing. We have this mantle we call the common law. We would really ideally like to wrap ourselves in the robes and wigs of the English judges. We are not able to do that so we come across with code words that assist us in that.

And in the common law roots of nuisance, we find that English law being concerned about things like a plague. . . .one of the first areas of nuisance, or the first area of nuisance was a field called a purpresture. Forget that word and forget I ever said it. It is of little use to you and you do not need to remember it. But a purpresture was essentially an interference with the public right. If you built a wall across a public highway, you were guilty of a purpresture. You were interfering with one of the King's rights, which was freedom of movement across a public highway. And in feudal times the King could emerse your lands. You would be at the mercy of the King and he would take your lands.

This was all done by the criminal law. It was not done in the fields that were called equity or chancery. It was done by the criminal law. You were prosecuted for the creation of a nuisance. And part of the prosecution was to put you in jail and to force you to abate the nuisance. There was nothing about clear and convincing or anything else. If you caused a problem, you were off to jail, your lands were at the mercy of the King — you were in real trouble.

Now I suggest to you that the development of purpresture then went into the fields of public health. You may find in the roots of English common law references to abatement through the criminal process of the building of tenements. And why? Not because the tenements caused disease, not because disease was found in the tenements, but because of the common sense and public health recognition that in medieval times when you packed people together in unsanitary conditions, the likelihood of disease was substantially increased. It was a risk that was created which would be abated. It was risk abatement, not disease abatement, but public health risk containment. So you find that at the very origins of the common law.

You find in the definition of common law nuisance again, these are public nuisances now. I am not going to bore you with a very fundamental, and both theoretical and practical, distinction between public and private nuisances. But most of the law that has been developed with respect to private nuisances has only sought and resulted in the confusion as to what our opportunities are under public law and the law of public nuisance.

There is a description, for example, of a public nuisance of "one who carries a child infected with the pox along a public highway or adjacent to populated dwellings." Again, the risk of infection is there. The risk is what creates the public nuisance subject to criminal abatement.

The reason that I wanted to take you up to this point is to show you not only the principle has been established early in the law but also to show you where the proof comes in.

When you go before a judge today and you want a discharge source stopped, the judge is exercising what is known as the jurisdiction of the Chancellor, the jurisdiction of courts of equity. In England there were divisions between the chancery courts and the courts of law. The chancery courts were able to effectuate rapid relief, immediate relief, immediate abatement of a problem. One of the things that grew out of the Magna Charta was the right to trial by jury.

As you know, today, we lawyers are famous for our ability to delay proceedings. Well, the lawyers in those times were also capable of delay; and the trial by jury resulted in inordinate delays. So many times the public prosecutor would come to the chancery courts and say, "We don't have time to wait for trial by jury. We don't have time to wait for the facts to be found by a jury of our peers or a jury of the defendant's peers. We must have immediate relief."

You see this articulated repeatedly under such people like Story, who was a famous justice of our United States Supreme Court. The equity courts would say, "Well, look, when the facts are in conflict, when there is Expert A, Dr. Jones, and Expert B, Dr. Smith, in conflict over whether or not this really is a hazard, I will demand of you clear and convincing evidence. And where I have this scientific conflict, I will very often deny relief because of the very conflict in testimony."

Most lawyers have stopped there. They have said, "If you want common law public nuisance relief, you must show clear and convincing evidence, uncontroverted evidence or you will not get the relief." They never bothered to read the procedure thereafter.

The procedure of the courts of equity historically was to then send the case back over to the jury calendar in the law courts who would then try the case, find the facts, and then upon the finding of a hazard, relief would be granted. What we have done today is to have eliminated this time-honored distinction between law and equity.

The concept of clear and convincing evidence of a hazard, of clear and convincing evidence of actual harm, for example, that has existed in equity at best continues in the field of what is known as "preliminary injunctive relief" where we go into court tomorrow and seek an immediate injunction. It has no historical or practical basis in the area of common law nuisance or any other area in dealing with a fully litigated controversy.

I suggest to you as scientists to read a decision which is both a basis of support for my position as well as one that creates great food for thought. It is the case of Ethyl Corporation versus EPA. It has nothing to do with nuisance, but it has a great deal to do with the whole question of scientific probabilities in the field of judicial review of administrative decision making as well as the whole question of judicial decision making on the facts. Judge Skelly Wright is the author of the principal opinion, and it deals with this whole question of lead additives in gasoline and the kind of public health hazard that they presented.

What I am saying to you is that public health hazards, if clearly articulated, even in the presence of conflicting or contradictory evidence on the other side, are the proper subject of judicial scrutiny and judicial action on the basis of the preponderance of the evidence. Preponderance of the evidence simply means: there is evidence on one side that says one thing; there is evidence on the other that says the other—which am I most convinced by? If I am convinced by the experts on public health hazards, then I am going to go forward and I am going to grant relief.

One of the things that is very, very important in this area is that in the public health field, the concept of balancing conveniences is looked upon with a great deal of resistance. The concept of allowing the public to be exposed to a significant health hazard simply because it is convenient for the discharger to continue discharging is looked upon with great jaundice by the courts.

One of the cases that I think you should be most familiar with in the whole field of health hazard protection and environmental law is the case of Reserve Mining versus United States. Reserve Mining versus United States is a classic example of how these things develop and how the law handles them. I have to say on balance that it was a very good decision. I disagree with some of the law that was used, but it was a very good decision ultimately in terms of the result.

Judges lately have created this doctrine and they think it is new; they think it is something they invented. They call it risk-benefit. They say, "What is the probability of the harm? Is it great? Is it small? What is the statistical likelihood of the harm occurring?" And then, assuming we have established the statistical likelihood: "What is the severity of the harm if that were to occur?" It is a two-pronged test. It could have a very low statistical probability but great harm if it occurred. It could have a very high statistical probability but be innocuous if it occurs.

Among the first cases to articulate the question of risk-benefit is the Reserve Mining case and Judge

Skelly Wright. And it has been articulated primarily in justifying EPA or OSHA or Department of Labor regulatory action in such toxic substances or potentially toxic substances as aldrin-dieldrin. It has been involved in the field of asbestos in AFL-CIO versus Hodson. It has been involved in the field of vinyl chloride control in an OSHA regulation. And, in a nonregulatory sense, in a courtroom sense, it was involved in the Reserve Mining case in the field of asbestos-like fibers.

And I suggest that you all, if you get a chance, wade through the final decision of the 8th Circuit in the Reserve Mining Case because it shows you how far the courts will go with the appropriate public momentum behind them. This is a very important aspect. Courts do not operate in vacuums. Courts are responsive to the public mood; the courts are responsive to the political winds just like anybody else. But they will go very far forward in protecting the public health.

In the Reserve Mining case, first of all, it started out as a water pollution case from the standpoint of: Was Reserve Mining discharge causing anything with respect to the eutrophication or with respect to the fish or the biota of Lake Superior? I do recall having worked with a number of EPA scientists who were involved with that case and were working with me in related cases. There was a great deal of frustration about their ability to prove or disprove the question of water quality impact.

Then along came public health. We lawyers always look for this. I mean we have to have it. It's a blood and guts issue. No question about it. Public health has sex appeal. Public health is something a judge or any decision maker can relate to. Very few decision makers want to be subject to cancer. Very few decision makers want to be subject to a debilitating disease. So if you can say to them, "Look, you or your children or your relatives (or whatever the case may be—you do not say that to them directly but the level of analysis is there—they are thinking this) are clearly going to be subject to a serious health risk," you have come a long way in establishing the basis for abatement of a discharge.

Here you had Reserve Mining's asbestos-like fibers—not clearly asbestos, not proven to have all of the properties of asbestos, subject to a great deal of conflicting evidence as to the mineral and geologic composition of these materials as to their properties of being like asbestos, but nevertheless accepted by the court as having the same properties as asbestos. All of the evidence with respect to harm caused by asbestos relates to inhaling asbestos—ingestion through the air. There is absolutely not one iota of evidence with respect to the carcinogenic effects of oral ingestion of asbestos through water. There is no epidemiological evidence, no clinical studies, nothing. It is unlikely that epidemiology could ever develop that evidence.

The court, after working through all of this and working through the absence of evidence on many of these subjects, essentially relied upon the statements by the medical and other experts testifying in favor of control, that yes, in their opinion there was a public health risk. It was impossible to quantify the degree of the risk. It was impossible to quantify—and I defy anybody in this room to ever quantify—assuming that cancer could be contracted, how many people would contract cancer, what the degree of the severity of the cancer would be, what the ability to cure the cancer would be, what the susceptibility to therapy would be, what the ultimate death rate would be, what the degree of pain would be. The impacts of these kinds of health risks are impossible to quantify.

One of the things I want to disabuse anybody in this room about now is the game of trying to put costs and benefits on these risks. You cannot do it. I would love to cross-examine any expert who professes to be capable of doing it, to simply say, "Well, this risk we will throw out the window; we will not control because it's only worth $27.50 and to control it will cost $36.42, therefore, the costs of control outweigh the benefits of control and therefore we should not do it." This is a popular game that is going on these days. We have other words for it, cost-effectiveness, whatever the case may be. I suggest to you that it is impossible. When all of the costs are laid out in the equation, and I can always find costs that nobody else can because there are experts who have not been brought into the equation, there are variables that have not been brought into the equation. Then when you start cross-examining you will say, "Well, yes, I didn't include that. Yes, I did know about it but I didn't think it was pertinent." It is always elements of risks, elements of hazards that are ignored in this process. So I suggest to you if you think we are going to play some nice little game that we can plug into a computer and come out with that lovely word "matrix." They love matrix. Matrices. I go crazy over matrices.

You are crazy. You cannot quantify these things. You have to lay it out on the table. You have to lay out the areas you do not know. You have to do it honestly. And then it basically comes down to, yes, there is a risk—nothing more, nothing less than that.

And the court said, "We cannot quantify degree of risk. We cannot quantify the likelihood of degree of injury assuming the risk takes place, assuming the statistical probability does actually occur. But we have identified a risk and we are going to cure it." They went the abatement route simply on that basis.

And where did they find their legal hook? They said at the time, "We do not want to get into the murky waters of common law nuisance," which really are not that murky if you do a thorough analysis of it. "We are going to rely on language that no longer exists in the current Clean Water Act or the 1972 Amendment, the language that said where a discharge 'will endanger the public health.' " That was an interstate control mechanism that existed in the '65 Act that no longer exists, unfortunately, in the current amendments. But it was essentially identical statutory language to the public common law nuisance concepts that existed for 700 years—endangerment to the public health. You find it in Reserve Mining; you find it replete in the Milwaukee case.

In the Milwaukee case again, I ask you: Why am I typically in favor of the judicial process rather than the rule making process in this area? Because I have seen my ox gored more times than I care to think about by legislators or quasilegislators that say, "We decided against you." What evidence did you use to decide against me? "Well, there was some statement by so and so." What evidence did you use, what specific logical progression of analysis did you use to reach the conclusion? "Well, policy. It was policy." You cannot get straight answers. There is a crude two word Anglo-Saxon expletive that refers to what ultimately gets translated into policy by those who want to defend some of these decisions. But the fact is that there is no substitute on the basis of defining where the process of decision making went, on what it was based, where its weaknesses are. I will be the first to admit that many of the proposals that we put forward in litigation on many of these things have empirical holes. I have yet to come across a position yet that is totally free from empirical doubt. The day it is, hire me on—I will do it for free. The fact is that you have to honestly acknowledge the areas where there is doubt, where there are unknowns, and then develop it from that level of analysis. See where your paths of alternative action go. But deal with it honestly. Deal with it up front. Then the beauty of litigation is that you and the other side are forced to lay out your data, to lay out your methods of analysis, and to expose your thought processes to see how good you are, and how bad you are.

One of the cases that I am dealing with now concerns a little dam down the Mississippi called Lock and Dam 26. And we hired the Corps, whom I love. They are probably the best agency in the federal government. Why? Because they are unlike EPA. EPA is afraid of their own shadow. EPA will take its convictions and run around worried that its convictions will endanger its appropriations.

The Corps on the other hand is tough. They will look you straight in the eye and say, "We're robbing the public blind. We are going to destroy the environment. And what are you going to do about it?"

We have not done a very good job about it, I will tell you that straight up. The first thing the Corps told us in this Lock and Dam situation was cost benefit analysis (they are pros at that). Cost benefit says the economic benefits of replacing this lock and building this huge new lock and dam far outweigh the economic costs or the costs of construction which were a mere $400 million. We questioned that. We heard some comments that blew the Corps out of the water and just showed how their economic data was false, and just could not withstand critical analysis.

The Corps, when challenged, will always retreat to its bastion of expertise—engineering, civil engineering. "Well, at one time we did say the existing structure could be replaced at a relatively cheap cost. But now we find that it will cost more to fix up the old structure than to build a new one. It will cost $500 million to patch up the old structure and only $400 million to build a new one. Common sense says that we should build a new one."

What did we do? We hired the very man that the Corps and Bureau of Rec. and everybody else has hired around the country to look at dam safety. He came in, and he put in civil engineering terms, the very same phenomenon that you face in the scientific fields that you pursue, be it sanitary engineering or virology or bacteriology, you name it—and we do it in the law too. It is the phenomenon known as "black boxing." It is to put it all in a little black box and say. . . .pat the lawyers on the head—a dangerous practice, I warn you. . . .pat the lawyers on the head and say, "If you only understood, you would appreciate our position. But rely on us, please, because we're the experts. We know what we're doing."

And I have found time and time again that the critical basis of decision making can be distilled into language and levels of analysis that even the most stupid or ignorant layman can understand. If you cannot do that whether you want to or not, you have a major limitation on articulating your scientific position, because I defy you, whether you are a scientific advocate or a scientific decision maker, to be the source of all knowledge.

Many people have said, "Let's have an environmental court or an environmental decision making body made up of experts." Do you know what would happen? You, being a civil engineer or you name it, will find the only universal language that assists somehow in the communication of this sometimes is mathematics. But whatever your specialty is, you are going to come across fields you do not understand. You are going to come across fields you are unfamiliar with, or if you do have a knowledge of it, it is 30 years old. It is not current. But what you are going to do, because your ego is involved, is to be embarrassed about asking what seems to you to be stupid questions but may be indeed fundamental questions. And you will let slide statements that have no empirical foundation but which sound intelligent. But because you do not want to look stupid, you are going to let it go.

I remember, just as an anecdote, the dean of the law school I went to, who is a very famous man, has a habit of speaking in nonsentences. He is a philosopher. He would give dinner speeches. It was kind of like the emperor who wore no clothes. He would give dinner speeches that made not one ounce of sense. I would sit there with a puzzled look on my face while everybody would be nodding in admiration, oohing and aahing over this wonderful speech. I could not believe it. I would say after the dinner, "Did you understand a word of what he was saying?" And the replies would be: "Of course. Of course. Wasn't it the most lucid thing you'd ever heard?"

I suggest to you that is exactly the process that occurs when scientifically trained individuals have to decide upon their colleagues. They do not want to be embarrassed in front of their colleagues.

So I suggest again to you that the process is one of detailed analysis brought down and filtered down and distilled and analyzed from every perspective on the level of common sense communication and understanding. I suggest to you that the field of public law nuisance is an effective tool in dealing with these areas where Congress, in all of its wisdom, has hopefully left us a little room. I suggest to you also that when you are dealing with a proposal, whether it be a legislative or an administrative proposal, make your voices heard.

Now I do not know whether you folks in this room are in the field of selling mechanisms or chemicals involved in the destruction of pathogens. I do not know whether you are in this room from the standpoint of doing the analysis on it. I do know that I went through a four month trial with some of the top experts in the country—with Melnick and Berg and Sproul and Cliver and Wellings and you name it. It was like a convention. Walter Mack. And I learned a hell of a lot more than I ever wanted to know about viruses, a hell of a lot more than I ever wanted to know about bacteriology. Ed Geldreich was here from the agency. I suggest to you that I have tremendous respect for some of those people and their willingness to state what they do and do not know. I have a little bit less respect for the methods of advocacy of some others. But many of those people would say, "We don't know certain things and the level of knowledge that we have is imperfect. But what we do know is that we want to establish (I think this is a historical principle) what is known as the 'multiple barrier concept'."

I am just going to close with some basic facts in the field of pathogens that we developed out of the Milwaukee case. Anybody who says that an activated sludge secondary treatment plant is an effective mechanism for the removal of viruses does not know what they are talking about.

I have been buried for the last five years in the operating records of the city of Milwaukee. Assuming that they took accurate tests, which they did not. . . they have a little game that goes on down in Milwaukee. I use Milwaukee as an example. This is typical of most communities.

First of all, most of the sewage never gets there or a good chunk of it never gets to the plant. Why? Because they were building sewers where it is all gravity fed down to the plant, but they are building them as we say in the Midwest, ass-backwards. A 44 inch sewer would be downstream and then a 72 inch sewer would be upstream. A little common sense tells you what's going to happen. You are going to have overflows. Well, there are overflows all over the city. And this is not a practice that you can turn back to the turn of the century and say, "Oh, well, those poor dumb sanitary engineers like Metcalf and Eddy and all those people did not know what they were doing." Just so you know, the people who designed the original system for Milwaukee back in 1911 were Harrison P. Eddy and I forget who else was involved. But they were prominent sanitary engineers of the time.

We would get into situations where you would find memoranda that said. . . . .sorry, those of you who do or do not know the Milwaukee situation, they built this plant called South Shore. South Shore was supposed to relieve the problems at Jones Island. Except that Jones Island had a little thing that is a very profitable operation called milorganite. They do not want to send their high nitrogen wastes down to South Shore because that goes away in a sludge disposal and it does not go to the milorganite plant. You literally have the milorganite operation wagging the dog. It is "sell high nitrogen milorganite" that the whole goal of the Milwaukee systems gets to be.

But you would see a memo in the plant records that would say, "Sorry. In last night's storm I had to send you about a hundred million gallons. But I assure you, it never got there."

And it does not get there. The relief gates handle that problem. So that is one problem.

Now assuming it gets to the treatment plants, let me suggest to you again on the basis of the records, Milwaukee would know when the good days and the bad days were. By Thursday of the week, you were starting to get some bad days. The best day to test was on Monday. The industrial loads had not built up to the point where they were really affecting plant performance. Monday was your best day to do tests, because you would get some relief over the weekend. So all the tests that went to the state were basically based on Monday's results, particularly in the field of bacteriology.

That was a grab sample that was highly questionable as to how good it was and the methodology for taking the samples was also suspect. But assuming it was valid, you would have days at an activated sludge plant, which is supposedly a model for the nation, where the influent solids and the influent BOD would be 280, 300. And the effluent solids and effluent BOD would be 280, 300.

Maybe better.

So anybody could sit there, as one witness did, and say, "Oh, I looked at those records. That's 90 percent removal." It may be 90 percent average removal over a three year period, but it certainly is not day-by-day removal. That is what we are talking about in the field of public health protection. We are talking about the use of geometric averages which I find to be a fraud.

A geometric average, according to the scientists, takes into account the wide variability of test results. The fact is that if counts are coming out of a plant at 300,000 fecal coliforms per 100 ml after chlorination, something is wrong. And when a geometric average on the basis of monthly samples or monthly averaging allows you to report to the state that you are in compliance with EPA standards, something is wrong. I tell you that from a legal standpoint; I tell you that from having investigated it from the public health standpoint. If you are allowing instantaneous discharges (which we think to be a low number, parenthetically), something is wrong with your facility and something is wrong with the system of regulation which allows that kind of thing to happen.

I am suggesting to you that activated sludge plants are not an effective mechanism for viral control and, the way most activated sludge plants are operated in this country on a day-to-day or hour-to-hour variation basis, not a very effective mechanism for bacterial pathogen control. I suggest to you that EPA, if it is going to be talking about effective barrier protection on public health hazards, has to be looking first of all at effective collection. This is right now one of the biggest disgraces we have going in this country, both legally and technically, because everybody is ignoring it.

Most municipal systems are operating as combined sewers throughout the metropolitan areas of the country. EPA for political and financial reasons, and not for technical reasons, is ignoring it. I suggest that it has to be faced and faced right up front; and Congress is going to have to deal with it.

I am an AWT advocate—I am a fan of AWT—yet I will hear some sanitary engineers saying, "What the hell are we talking about AWT for when we're ignoring all of the rest of that stuff coming out?" And what they say makes quite a little bit of sense. To sit there and build an AWT plant when you have flows coming out of those overflows day in and day out is pound foolish.

Now my approach would be to attack both problems, not ignore one for the sake of the other, not to say, "Gee, that allows us to ignore the AWT responsibility." Attack the one and attack it honestly and attack it openly and not sweep it under the rug like the laws have been doing and like the administration of the program has been doing.

In administration, you cannot lay this at the administrative doorstep of EPA when the municipalities do not want the problem addressed because they know if it is honestly addressed, there is not enough EPA money to go around and they want to take the political and legal heat of controlling the problem. The next aspect is the congressmen. The congressmen do not want it addressed either. To me it is probably a private scandal, but it is one that has to be dealt with.

The next question is the flow control when it gets to the plant. I do not know of a major municipal plant in the country. I have to say that maybe some of the big MSD facilities can do it because they have so much reserve capacity. When you are talking about south-southwest in Chicago which as a 1,750,000,000 gallon a day capacity, they can get hit with a big flow slug and absorb it within their reserves. But most plants get hit with these highly variable flows. They do not know what the hell to do with it. The honest operators will tell you, "Well, there just went our sludge blankets out in the lake or the river," or whatever the case may be. And anybody who says under those circumstances that there is effective pathogen control is again whistling "Dixie." It just does not happen. So you have to have flow equalization.

And then, if anybody can articulate for this audience or me after the meeting tonight how the biological process within an activated sludge plant actually kills—kills, inactivates, whatever term you want to use, not just takes out by way of the sedimentation process but actually kills—viral pathogens (I do not know about the bacterial pathogens), then he ought to get a Nobel Prize, because the process is not understood. The way I understand the process to be from all the measurements I have seen, at best in an activated sludge plant, is that you are sedimenting the stuff out and then you can hopefully handle it through some kind of sludge treatment process. But you are not killing it by way of the activated sludge process, and it is going to go out into the receiving body if the activated sludge process acts up.

I am suggesting that there ought to be at treatment plants, once collection and flow equalization are attained, a third level of treatment which involves some physical process, either physical or physical-chemical process, for balancing out, evening out, the highly variable performance of activated sludge sewage treatment plants. I suggest to you that the incremental costs of doing that are not that significant. I have been surprised to find in a number of cases that I have worked on around the country that people are amazed when they talk about the total package for an AWT and then they are told what increment of that is secondary or activated sludge or biological. It turns out that well over 90 percent is typically in the secondary, and the so-called filtration or physical treatment is a relatively small (still a significant but a relatively small) increment.

So I am suggesting that we have much to do, much more. We have to do it by way of honest recognition of what the problems are. We cannot be going around the country saying that there is not a problem. I suggest that, those of you who feel that there is a problem, you have a professional responsibility to speak out, because the state agencies who are typically tied. . . .the tail that wags the dog in the whole process is the construction grants program. I suggest to you folks that the state agencies are like little pigs at their mother's side. "Who's going to give us the money? What do we have to do to get it?" They'll do anything—lie, cheat, steal, whatever the case may be.

If the blessing from on high says, "There shall be no disinfection," we do not need disinfection. No problems. The public be damned. There is no epidemiological evidence that anybody ever got sick.

Listen to a good epidemiologist tell you the horror stories about reporting mechanisms for most pathogenic or enteric diseases. We do not have a very effective system. The question that really has to be addressed is: Are we concerned about enteric pathogenic organisms? Are we concerned about transmission by the water route? This is a system of protection that was developed 80 years ago, and has worked, I think, very effectively to date. It needs addressing because the problems of population concentration are such that those lucky assimilative capacities that we have had are being sorely taxed, so we have to be more rigorous in our point source discharge protection.

Is it worth it to protect public health? If it is not worth it, then I think that the people who are advocating it ought to clearly state: "We are willing to take the risk. We are willing to take the gamble on you, the private citizen's health." I do not think that is being done today, and I do not think the advocates of public health protection are making their voices heard.

All I get out in the field talking to the state agencies are, "This is the route we are going. EPA from on high has disavowed the need for disinfection and disavowed the need for effective control of pathogens. This is the route we are going because it is most cost effective." Sure it is most cost effective. It is less dollars out of your pocket. Whether it is public health effective or not is a question I hope we never have to learn the answer to.

So I am suggesting to those of you in the audience who are public health professionals that you get about the business of beating the clarion call or whatever the case may be by making the public aware that there is a serious problem here that is not being addressed. I would suggest to you that all the law suits in the world will not help that problem. All the law suit will do is help focus attention on the problem. But until you, as professionals, get the word out, then the political sphere is not going to react to it. And I suggest that maybe the next time around when the Clean Water Amendments of 1983 come around, or whatever the case may be, we are going to have a logical process for addressing the question of pathogenic public health protection that can be done on a nationwide basis and not just on the basis of a long, drawnout fight in the case of Illinois versus Milwaukee.

I was asked tonight if I would open myself up to questions, and I foolishly said yes. If there are any questions about my preaching to you tonight, I will be glad to open myself up. Anybody?

QUESTION: Under the present judicial system, is it possible that an individual in a regulatory agency can be sued on a case, for example, a cost effectiveness statement made by a technical agency regardless of whether it is federal, state, or what have you?

MR. KARAGANIS: The question was, and I am going to paraphrase, is there potential for individual agency official responsibility for engaging in and making the wrong decision in one of these areas of cost-effective analysis? As the Corps would say, "Glad you asked that." They are wonderful. They have a public relations campaign that will blow your mind. With respect to potential liability of public officials for making the wrong judgments, this is an area that is becoming increasingly pursued in the field of dam safety—a small thing called the Teton Dam failure. The Bureau of Rec. wisely settled like mad. They went out there with a pocketful of cash as soon as they could and were settling claims left and right. So I do not know what will ever come out with respect to the Teton failure.

But I am dealing with one now. I was telling Frank at dinner that I was involved in Home State Mining controversy out in South Dakota a few years ago, and we had successful results in that. But while I was there, EPA was funding two sewage lagoons in the town of Spearfish, South Dakota. If you do not know Spearfish, that is the home of the Passion Play.

Anyway, Spearfish had these two sewage lagoons and the local farmers out there—please rely on the common sense of the people in this nation—the farmers out there would say, "Nope, that thing's never going to work. I've tried to build stock watering dams out here. The ground soaks it up just like that." They told the local city officials; they told the local engineers; they told the South Dakota Department of Environmental Protection; they told EPA. EPA in its infinite wisdom said, "Oh, yes, it will work. I have the little black book here that says it will work." "No, it's not going to work." "Oh, yes, it is. As a matter of fact, we'll load it with bentonite. We'll pave it over with bentonite and it will hold."

Now as it happens out there in Spearfish, they have a thing called gypsum that just is throughout the land. Gypsum is like putting sugar in your coffee. The minute that water hits it, it dissolves. It is just incredibly soluble. And lo and behold, if anybody bothered to check, the ground underneath the sewage lagoons was loaded with gypsum.

I remember being out there and ranchers out there would say, "My God, we are seeing a trickle of sewage coming up about a mile and a half away from the sewage lagoons." EPA would say, "No, that is not sewage—not from this facility." Well, in the fall of last year, or maybe the spring (I forget which), suddenly what the farmers had been saying all along happened. The bottom of the lagoon fell out—just literally fell out. The entire lagoon is emptying now and it is coming up on my client's property a mile and a half away. It is contaminating a considerable extent of the public water supply in the area and the stock water supply of the area.

Somebody asked me, "Is there potential professional liability on behalf of those officials?" I answered, "Yes, there is." I think that one of the things that EPA is going to have to start doing is a review, before they pass their hands over these construction grants so blithely, of what hazards have been created here and, if there is any risk, be it to property damage or public health or otherwise, state that there is the perspective of potential liability. I cannot say it is going to occur. I would suggest to you that that kind of analysis has to be done.

QUESTION: On the same order, if an industrial official can be hit personally as in a pollution situation, why, on a legal basis, can't the reverse be true? For example, a chemical company with bad water, dumps a big load. He gets hit by EPA both as a corporation and also as the president and chief executive officer. Cannot, because of a precedent, the same ruling go back to the regulatory agency?

MR. KARAGANIS: Well again, I want you to understand something. One of the EPA enforcement tools over the years has been to come riding in and say, "Thou shalt stop polluting and that will be an enforcement suit." And in the next breath give a grant to stop it. That was part of the settlement we discussed earlier. I only wish I had as thick a wallet as that. I could have been in good shape.

But having given the grant to stop the pollution, if that does not stop the pollution, if that causes a problem, then I think you are right. Now in this community of Spearfish, for example, these are farm folk. They are not sewage specialists. Their response to this whole problem has been, "We only did what EPA told us to do."

They happen to have a pretty savvy lawyer representing them—the same guy who represents Home State Mining Company. He is seriously thinking of going against, what we call "up the line," that is, going against the state officials who approved this thing; going against the engineering firm who designed it; and going against EPA officials who recommended it and told everybody that it would work.

So this potential liability is there. It has not been demonstrated yet. I do not want you to run out and buy insurance policies tomorrow, but I suggest to you that it ought to be a major consideration in the design of your review programs, and it ought to be a consideration that your legal counsel should get involved in as a precautionary measure.

One other thing that I find to be the case is that most agency people look upon lawyers as pests. Get them into your programs. I realize they are a pain. First of all, most lawyers are lazy and they do not want to understand, but get them to understand what you are doing. Educate them as to what you are trying to accomplish, what the elements of your program are, and then use them as your own staff. Get them to check over what you are doing. Do not just send over a grant application or send it over to regional counsel's office or whatever and say, "What do you think of this?" because most of those guys do not know what is going on. They never have and they really do not want to know what is going on. But the ones who are trained, the ones that you will take the time to train, you will find they will be conscientious and will do a damned good job for you in covering you against the very problem we are talking about. If asked to do it, they will be the ones who will ask the hard questions. You have to do that internally before somebody like me gets hired to do it externally.

QUESTION: Probably the best thing that is happening then is for EPA to delegate to the states, just like it just did to Illinois.

MR. KARAGANIS: I just responded to a question like that recently. The question asked if probably the best thing was for EPA to delegate it to the states. I responded to that question with this answer which applies to this question as well as one I recently had. If you believe that, I have a used car outside in the parking lot that I would like to sell you. You have certain responsibilities you cannot delegate. Whether it be under the 201 program, 401 program, whatever lingo you want to use, 208, et cetera, I do not think you can delegate liability protection. You have an oversight responsibility here. You are dealing with your involvement of federal trust. You are the conservator of the federal law both in terms of grants, in terms of permits, in terms of enforcement, in terms of standards, whatever the case may be.

I have this little problem out in Las Vegas, Nevada, right now where EPA is running around like mad saying it is the state that did it. It was not the state that did it. The state is going around like mad saying EPA told us to do it, and they are probably right. They did not do it the right way, but you simply cannot get away with it by saying, "Thou shalt follow the law." You have a responsibility to make sure that whoever you delegate it to actually does indeed do the job. I am not telling you anything that the lawyers down in Waterside Mall do not know about, because they write the regulations and that says, "Thou shalt not delegate without checking up to see if it is done." You cannot get away with it in that process.

I really feel that many of these problems are coming up because there is that wholesale delegation without adequate oversight. I want to say that any of my remarks here tonight are done with the recognition that there are tremendous manpower problems. You have enormous programs to run with too few individuals. There ought to be some kind of mechanism for increasing the efficiency of individuals, developing safeguard check lists which move these things through, hopefully at a rapid administrative rate, but at the same time making sure that not too many mistakes slip through the cracks.

QUESTION: Have you done anything to challenge the coliform standard? This has been coming up in the conference—whether or not the standards are valid for environment and wastewater.

MR. KARAGANIS: No, I have not for a very simple reason. My ability to do anything is severely limited by the same thing that limits most of you folks—money. I have to have experts behind me; I have to have resources behind me. I have to have somebody that pushes the button in the back of my head and says, "Go do it." Until I can have that resource, I just deal with the specific problem. I am familiar with the field. I think it ought to be challenged.

Let me tell you about the so-called 200 geometric mean. In checking with Ed Geldreich, who I am sure has had a hand in setting the 200 geometric mean, he tells me the reason 200 was selected: "Well, at 200, we found a 7 percent likelihood of bacterial pathogen occurrence in the samples we analyzed." Nobody ever attempted to correlate viruses. Those attempts that have been made have shown there is an inconsistent correlation—that it is not a positive correlation. But at least the bacterial pathogen, is 7 percent. I do not know how many samples Ed did to obtain this. But when talking to him and hearing his testimony on the stand, he agreed that the geometric mean was inappropriate because the geometric mean would mask individual slug discharges. I think the use of the geometric mean is an illusion. It is almost a fraud to say to the public: "Okay, here at our bathing beaches if you go out swimming, one day out of the month we made it." The public buys that. I really think that it has to be laid out. Most newspaper reporters buy it. Who cares — geometric mean, arithmetic average, whatever the case may be. They just see a bottom line number. You tell them you meant it, and they think they are home free.

I think it should be attacked. I think it should be attacked professionally within the agency. I think one of the things that might be helpful within the agency is the articulation of responsible dissent. Where you have a policy that is up top, there ought to be an ombudsman mechanism or whatever else that gives responsible scientific dissent a mechanism for voicing its position so that it can ultimately be made public or somehow be made available to the public.

DR. REYNOLDS: You talked about public health risk. It seems to me that what you seem to be telling us is that, with the current standards, we have accepted a given level of risk which you are not willing to accept. It seems to me this whole issue seems to be: What level of risk is the public willing to pay for in terms of a public health nuisance?

MR. KARAGANIS: Yes, sir. Would you kindly tell me how many pathogenic bacteria I am exposed to at a level of 200 fecal coliform per 100 ml?

DR. REYNOLDS: I understand what you are saying. I cannot give you that answer, as you know.

MR. KARAGANIS: Not only do I know, sir, but the public has never been given that answer. When the public is told the standard of safety is 200 fecal coliform per 100 ml, they are not told that they have no idea how many pathogens are present or the fact that at any series of readings at 200 or below, any number of bacterial pathogens may be present or the fact that considerable quantities of viral pathogens may be present. The number has had no relationship to the viral aspects of public health and has had little empirical relationship to the standard of bacterial pathogens in public health. I suggest to you that from the standpoint of fecal coliform, I am just telling you as a lawyer what the experts have told me.

There is a guy running around saying, "It is all the birds." It is the birds or the fish or whatever the case may be. I tell you, we had big charts in Milwaukee saying that any fecal coliform found anywhere in Lake Michigan could come from 37,000 different sources, only one of which is human. I am told again within the limits of knowledge there is this (Geldreich has suggested and others have suggested) fecal coliform/fecal strep relationship, whatever the case may be.

But if we are looking for scientific analogies or methods of indication and if fecal coliforms are not satisfactory because of the possibility of interference, if there can be other indicator organisms that are more precise, so be it. If there can be other indicator organisms that would match in pairs between fecal coliform and fecal strep, so be it. But then once we do that, I think we ought to have a standard that says "as close to zero human fecal contamination as possible."

I do know that with current technology, and not at tremendous cost, most municipal sewage treatment plants can produce an effluent of zero fecal coliform. And if all of the sewage gets to the sewage treatment plant and if the sewage treatment plants can produce it, we should not find any fecal coliform with a fecal strep human source relationship in any public water. We will find fecal coliform from the ducks, and the fish and the birds and whatever; but we will not find it from humans.

Again what I am told is the diseases we are worried about and the fecal coliform which we use as an indicator is from human enteric sources. If we collect and if we treat properly, we should be willing to say that fecal coliforms or indicators of pathogenic contamination from human sources should register zero, because right now nobody can say beyond that exactly what the hell you are getting. In the virology field, you folks have to get your act together from a policy standpoint.

In the viral field, it is a massive fraud to the public to use that kind of an indicator and say, "It is okay." It's just not.

There is one regulation that is being inconsistently used around the country in terms of public health and safety. We have an Illinois Department of Public Health regulation that says: "No more than 20 per 100 ml for two days running." You would not believe the number of violations. I do not know what its empirical base is, but you would not believe the number of violations that hit at that standard that just slide right by the geometric mean—just slide right by. We are keeping our beaches open when they may be subject to pathogenic contamination.

I will buy all of the business that, yes, it has to get in; the dosage has to be properly adjusted; to establish a minimum infective dose for typical individuals, susceptible individuals, the whole business. What if it gets in and causes a disease? So what? Somebody is discomforted for a few days. You can go through all of those arguments, and I can find countervailing policies as to why you should protect because the risks are so great and the cumulative or additive effects of those diseases can be so debilitating and even life threatening.

Lawyers have developed with some justification—they have an axe to grind. They are being paid to do it. The only way I can be effective is, as Frank suggested, as a translator— someone who is articulating for laymen and assists experts in articulating their position. But it has to be the experts that are out in the forefront. It has to be the experts who are publishing the documents. It has to be the agency who says, "Cost-effectiveness—let's recognize it for what it is."

We are waiting out the Milwaukee situation right now. Milwaukee said, "We will agree to abate our overflows." The judge and everybody thought that was great. "We will abate them to that level which is cost-effective." What level is that? "We do not know." What level do you think it is? "Just so we cannot find any observable effect." Does the fact that you cannot find any observable effect mean that you are not causing any harm? "Well, no."

One of the things that has happened in the Milwaukee situation, concerns the Howard Street water intake. Can you imagine living in this situation? I cannot. I find it mind-boggling. The expert from Milwaukee called up the water treatment plant thinking he was going to find evidence of how clean Milwaukee's water is. And one of the big problems we have in this field is the damn water treatment plants are still doing total coliforms and not doing fecal coliforms. And the sewage plants are doing fecal coliforms and not doing total coliforms. Try and find out that statistical relationship sometime in a cross-examination.

Now you have to get some consistency in the methodology that you are demanding as well. I say do them both. If you are checking your water, do them both and do them the right way as well. One of the things you are still doing is following this quick and dirty method of the membrane filter test which is causing so many problems. Milwaukee found it was unusable. Milwaukee found it was giving them low numbers when the MPN test was giving them astronomical numbers. You have an honest bacteriologist in the Milwaukee sewage lab saying, "Let's stop using the MF method. It's giving people the illusion that the waters are safe," because the Milwaukee Health Department continues to this day to use the MF method. The sewage people are off doing the MPN test and finding incredible numbers. I will not go through the long list of interferences which many of you are familiar with.

The MF test is fine if the water is clean. If the water is dirty, it is not good. So you use it to find if there is dirty water? Let's get some consistency. I always look for the thread of common sense that is running through these things and sometimes I do not find it. We are all working towards the same goal. Let's sit down and get some plans together.

But this is a problem of logical consistency and planning. At Milwaukee, do you know how Howard Edner responds? Everytime it rains, they dump carloads of chlorine in the intake. They just dump like mad.

Now people tell me, and Milwaukee told me, "My God, you can't use chlorine because chlorine causes carcinogenic compounds to form which may cause a problem." And indeed it may. One of the ways to do it is not assault the treatment plant, the water intake plant, with pathogenic loads so that you have to use that much chlorine.

One of the things that we have found through sewage treatment and otherwise, is that the cleaner you make your effluent the less chlorine you need to do an effective job.

Believe me, if there is an effective way that has been established and will be accepted by EPA to use something other than chlorine which will resolve the questions that have been raised about chlorine, I am all for it. But we looked because we wanted to impose the most effective mechanism for disinfection in the Milwaukee situation. The only one that all of our people told us that we could have some certainty on at that stage of the game (that was a year ago) was chlorination. If you can come up with an effective thing for large treatment plants to use other methodology, fine. Let's get into it and let's resolve the questions about chlorine. But even with chlorine, you get the organics out and you get the solids out. You convert the ammonia and you are in good shape with respect to being able to do an effective job of disinfection with minimal development of chlorine compounds that could be potentially carcinogenic.

All the questions you raise are legitimate ones. But you put them out on a table and subject them to analysis and you will find that many of the things that the experts are telling the public just are misleading as hell right now.

Anybody else? Yes, sir.

QUESTION: One word in favor of using the geometric mean. It has evolved not to evade standards, but because bacterial populations grow according to a geometric rate. Therefore, in order to find the correct average, to get a straight line on a graph, you plot it on a log-log graph and take your midpoint. So there was no intent by any biologist to evade regulations.

MR. KARAGANIS: I know that. But you see, laymen take your tests. Maybe your test and your geometric mean may have perfectly valid scientific rationale and may be useful within the context of your scientific analysis, but a layman will distort it. A layman will take it and run with it. A lawyer will take it and run with it and say, "My God, I have a test I can use." He puts it in and it is misleading. Most public health people that I have talked to want to know about the worst case. They do not want to know about averages, because averages are misleading. They want to know about the worst case—what is the worst potential case. And the geometric mean from the standpoint of receiving waters does mask. It may be fine for the laboratory basis, but it masks worst case situations in receiving waters. That is the only point I am making about geometric mean.

And if you tell me that you get a high result in the bacteriology and it may be because of a lab error, it may be because of anomoly as to the way that bacterial grew and there really were not X thousand bacteria out there, I will buy that too because the tests are crude, the tests are ineffective, the tests very often will give you erroneous results. But from a public health standpoint, recognizing all those limitations, if the test shows up with a bad number, I want to be able to prevent, I want to be able to see that the situation does not develop. And that is the kind of protection we have to work for.

If there can be more effective tests than MPN MF, whatever the case may be, so be it. If there can be effective viral tests, so be it. But within the crude limits of knowledge we have today, let's maximize public health protection.

APPENDIX B.

SYMPOSIUM RAPPORTEUR

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One of the best things about the symposium, in my opinion, has been the transfer of information and the audience participation. When I first came here, I figured this was too large a crowd. You will not get two people to get up and say anything besides George White.

But it has been far better than that. I have seen numerous people get up and make comments. I really think that is good and I am glad to see that Al incorporated this into the program.

I guess one of the distracting things is the incompleteness of several of the studies. This is invariable when you have projects that are in various stages of completion. You cannot expect not to find some of these things. When you are organizing a symposium you cannot wait until everyone finishes. If you did, you would never present anything.

There was a limited amount of practical, how-to-do-it kinds of things presented too. I heard several people comment about that during the symposium. I think perhaps the next one, after some of these studies are finished, will solve that problem. I certainly hope so. I do not mean to imply that there were not any practical presentations—certainly there were some. But I think that was probably one of the limiting factors.

I think the EPA Research Program is addressing the problems that we face. They found some solutions, but I think in most cases though, they have asked more questions than they have answered. That is not unusual either for research projects. If anyone in this room has ever conducted a research project, they know you wind up asking more questions than you ever answer. But as long as the intentions of the agency are to try to continue to answer these questions, then that is fine. We have accomplished our purpose of being here..

I will make a few specific comments about some of the questions that came up during the symposium. The issue of using hypochlorite was mentioned, and people 'talked about hypochlorite generators. There is a simple technique for small systems that appears to work very well, at least in the one instance where I have taken any data. This is simply a tablet type feeder where you pass water over tablets in a tank and allow it to go into a contact tank. The only one that I mentioned that I am familiar with is located in a small town in New Mexico, and they are chlorinating lagoon effluent. It does a good job, and they have very low coliform concentrations coming out of the lagoon system which they then filter. It is an excellent quality effluent following that.

That is something, if you are dealing with small systems, you might keep in mind. I am sure it is not the most effective use of the hypochlorite; but nevertheless, it does not require a great deal of talent to keep it running. If you see tablets in there, it is running. Most operators are capable of handling that.

There is a great deal of discussion about regrowth of coliforms. I do not think anyone reached any conclusions as to what was actually happening.

I think one of the biggest problems that I detected, in addition to the few things I mentioned before, is that we do not know what we are measuring half the time. We talk about the UV, we talk about the coliform, we talk about the various residual techniques. Now maybe in the future we will be able to solve that problem, and I saw some indication of that during the course of the meeting. People are beginning to talk to each other and there were some comments by the standard methods people and the other standards organizations. I hope we can get around to that because half the time we are not talking about the same thing. Someone disagrees and yet, he does not even know what he is disagreeing with. I think that is another issue that should be pursued very actively in the EPA Research Program.

The fish bioassay was also presented as a method of evaluating the impact of the various by-products of disinfection. It is certainly a good technique. There is nothing wrong with fish bioassays. But you cannot find many people in this country, including most of the scientists, that can run a decent fish bioassay. If you have ever tried to keep those little beasts alive, you will know what I am talking about. They just get all kinds of diseases and they do not pay much attention to what you do. You can put all kinds of streptomycin in there and everything else, and they belly-up on you. So the fish bioassay is a tough thing to run.

Maybe we need something simpler for the operator to detect these types of things. I know one thing that may or may not apply, but it does not sound like a bad idea to me, and that is the simple bottled bioassay technique that has been used in eutrophication studies. It is about as simple a test as you can get with the biological organism. It does have the advantage of simplicity, and I know many operators that can run it. Plus or minus 50-60 percent is not bad considering most bioassayists that you run into. I think that again is something that might be considered in the future research program.

Last night we heard a great deal about standards and the risks that are involved and associated with certain levels of fecal coliform, or whatever we are measuring. Regardless of how clean we make sewage or any other waste material that we are destroying, there is going to be some risk associated with the discharge of that product. Contrary to what anyone wants to say, there is some risk just sitting in this room. We have to decide what risk we are willing to accept. I think to run out and do something simply because you can do it is foolish. I do not think that is what was being said last night, but I could certainly get that implication occasionally. I do not think that is very smart. I think it is extremely foolish in terms of fiscal responsibility. We should decide what reasonable risks are and proceed from there. I know a lot of people do not like that, but when you start looking at some of the other problems in life, it certainly has to be dealt with on a risk-analysis type basis.

We might take the same approach as the people who are dealing with the problem of the dam now. They have been working basically with stochastic type processes, and it is something that we might be able to apply to disinfection and control of diseases. I am not sure, but it would be worth a shot.

There is obviously a great need to do a lot more work concerning the side effects of the various type of disinfectants. I think if nothing else came out of this morning's session, it certainly shows that we do not know much about what we are doing and much about what the side effects are of any of the disinfectants. We know more about chlorine perhaps than we do the others, but I am not sure we know much about any of them when you really get down to a basic situation. I think that is something that we could definitely investigate and improve. I know there is work going on in that area. I hope there will be more.

The cost figures obviously need to be coordinated. I think we are all at fault when we present cost figures. We invariably wind up leaving out things because it is convenient or we want to put things in and make somebody else look bad. We go through a lot of procedures and, although not always intentionally, we still make a lot of mistakes. I think it would be nice if the manufacturers would be the first. Don't wait until somebody beats you over the head or embarrassses you. Be first and lay those figures out. I think you were today and yesterday in trying to get your figures out, and it was certainly an eye-opener to me because I have seen basically the same kinds of figures that Ed Opatken had presented before. It is nice to know that things are being improved and they are coming down in cost. I think the best thing to do is to contact someone with experience. If they know what they are doing, they can certainly help you a lot when you start looking at costs.

Two other little quick items and I will quit. I would like to caution you against using some of the general formulations that you saw. They were not presented as general formulations. They were presented as specific formulations. For example, Reynolds' and my work certainly should not be used indiscriminately. You have to use it carefully, and the same goes for any of the other formulations that you have seen. Make sure you know what you have before you start using them. You can get in more trouble than you would care to have.

I think the routine monitoring techniques, which I alluded to a little earlier, need to be improved and some of the automatic feedback control type devices need to be improved. Once those come into play, I think we will have much better acceptance of some of the newer technologies.

One item that was briefly mentioned, but very little was said about it, is our energy crunch. I think if things stay as they are now and the Arabs continue to jack up their oil prices and Congress continues to drag their feet in coming up with some decent energy policy, that is something that is going to have a tremendous impact on disinfection and what process is accepted.